VOLUME 85 NO. SM5
OCTOBER 1959
PART 1

JOURNAL of the

Soil Mechanics and Foundations Division

PROCEEDINGS OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

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Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

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October, 1959

Papers

(Over)

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

A STATISTICAL STUDY OF SOIL SAMPLING

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SYNOPSIS

This is a study undertaken to determine the number of samples needed to obtain reasonable correlations between pedologic soil types and their engineering properties. Data from four DeWitt County soils give a quantitative indication of the value of pedologic information in planning, designing and constructing highways and airports in Illinois.

INTRODUCTION

The problem of planning an adequate yet economical program of soil exploration is one of great importance in the field of soil engineering. Many engineers are constantly striving to improve their attack on the problem by utilizing available knowledge of the local soil situation. Such knowledge may be gained from geologic reports, pedologic reports or construction experience. The construction area covered by most highway and airport projects involves many acres. Consequently, for such projects it is practically impossible for the soils engineer to follow as detailed a program of soil exploration as is normally conducted for structures covering a more limited area.

In order to increase the effectiveness of areal soil surveys many engineers rely on pedologic (sometimes called agricultural) soil maps and reports as an aid in planning the field program.(1,2,3) Since 1952 an investigation of the engineering uses of these maps has been conducted as project: "Soil Exploration and Mapping" of the Illinois Cooperative Highway Research Program.

A principal objective of this project is the determination of the physical characteristics of the various pedologic soil types mapped in Illinois. The

Note: Discussion open until March 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2210 is part of the copyrighted Journal of the Soil Mechanics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 5, October, 1959.

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attainment of this objective requires the testing of samples from each major horizon of every important soil type. In the summer of 1952 a comprehensive program of soil profile sampling was begun in DeWitt County in east central Illinois. It was hoped that the results of this program would reveal the average physical properties of each pedologic map unit as well as the normal deviations from the average. In initiating the sampling program it was necessary to specify the number of profiles of each type which should be examined in order to establish the values of these properties. For reasons of expediency it was arbitrarily decided to sample 3 profiles of each soil type mapped in the county. In the case of the four most extensive soil types, 4 profiles were sampled.

It was recognized that there was no sound basis for choosing these particular numbers of profiles for sampling; this fact led to the consideration of statistical methods of control. If it were not for variation between profiles of a given soil type, then samples from a single profile of each type would be sufficient. The problem of selecting the necessary number of samples is complicated by the fact that all physical attributes of the soil profile do not vary to the same degree. For example, there may be considerably more variation between the liquid limit values than between the silt contents of samples taken from the same profiles. Thus, the problem can be solved only in respect to the number of profiles required to determine the average liquid limit (or other specific property) of the A horizon (or B horizon, etc.). Even this solution can not be obtained until a decision is made with regard to the degree of precision desired. This decision involves a statement regarding the odds that the mean of the population will fall within a specified range.

The foregoing discussion indicates that there can be no definite solution to the sampling problem unless certain limitations are accepted. Some of these limitations are imposed by the nature of the data, others by the investigator on the basis of his judgment. The initial DeWitt County sampling program indicated that with regard to certain index properties some soil types were quite variable whereas others were relatively uniform. It was felt, however, that 3 or 4 soil profiles did not provide sufficient data for a satisfactory statistical analysis. For this reason a supplementary study of the four major soil types in the county was undertaken in 1954. Samples were collected from the A and B horizons of ten additional profiles of these soil types. Routine classification tests were performed on these samples and the data obtained from the ten profiles was subjected to statistical analysis.

Three of the soil types included in this study were developed from a loess cover 40 to 60 in. thick overlying early Wisconsin (Tazewell) glacial till of loam to silt loam texture. The fourth is a depressional soil derived from the local wash in the same area.[6] Flanagan silt loam could be considered as the typical soil of the area. It has been developed under prairie grass vegetation on slopes varying from 1 to 4%. The total thickness of the solum seldom exceeds 40 in. Catlin silt loam is formed on the rolling portions of the till plain and moraines where the slopes vary from 3 to 7%. The solum is entirely developed from the loess cover under prairie vegetation. Birkbeck silt loam is a correlative of the Flanagan and Catlin developed under deciduous hardwood forest vegetation on slopes varying from 2 to 8%. The solum may vary up to 40 in. thick and is nearly always derived from the loess; however, calcareous till is likely to be found a few inches below the B horizon. The depressional soil, Drummer silty clay loam, is developed on slopes of 0 to 0.5% under slough grass vegetation. The surface is high in organic matter and the B

horizon is often poorly developed. The solum has been developed from moderately heavy-textured water-deposited sediments and is underlain by sediments varying from sandy loam and loam to silt loam and occasionally silty clay loam. Either sandy outwash or loam till may be encountered at depths below about 5 ft.

Statistical Study

In both the original and supplementary studies the sites for sampling were first located on the pedologic soil map.(4) The sites were selected at random although some control was exercised in order to prevent concentrated sampling in a limited area. The average length of DeWitt County in an east-west direction is about 25 miles. A major portion of the western half of the county is occupied by the Shelbyville moraine. Five profiles of each type were sampled in the morainic area. The other five were obtained in the Shelbyville till plain area which occupies most of the eastern half of the county. Within each of these two areas a center of sampling was located by chance and wherever possible one profile of each of the four soil types was picked within a radius of about 2 miles from the center. If it was not possible to locate a sampling site for a given type within this radium then a new center was chosen by chance for sampling this type. This process of selection was conducted in the office so that field conditions had no influence on the choice of sites. In the field a final check was made only to assure that each site met the criteria of topography and drainage set forth in the typical soil type description.(6) However, no conscious effort was made to choose average or "typical" slope conditions. In view of the sampling method which was followed, it was felt that the data were suitable for statistical analysis.(5)

The values of three index properties were chosen for statistical study. These were the liquid limit, the plasticity index, and the minus 0.002 mm clay content. Table 1 gives a summary of the basic test data and the statistical data obtained. The first three columns identify the index property, soil type and horizon. Columns 4, 5, and 6 give the minimum, the maximum and the mean values obtained from the tests on the ten samples from each horizon.

A commonly accepted measure of the dispersion of data about the mean is the standard deviation given in Col. 7. In a normally-distributed population approximately 68% of the individual values will fall within $\pm$ 1 standard deviation of the mean, and 95% within $\pm$ 2 standard deviations of the mean. If an adequate number of random samples are taken from a population having a normal-type distribution, the standard deviation of the samples may be used as an indicator of the range of values that exists in the population. For example, it may be estimated that the liquid limits of 95% of all samples taken from the A horizon of Birkbeck silt loam in DeWitt County will fall within the range $37.4 \pm 2.26 \times 5.2\%$, or from 25.7 to 49.1%. For comparison, the liquid limits of 95% of all samples taken from the A horizon of Drummer silty clay loam should fall within the range $54.6 \pm 2.26 \times 3.8\%$ or from 46.0 to 63.2%. The multiplier 2.26 must be used instead of 2.0 since the reliability of the standard deviation of the samples as an indicator varies with the number of samples analysed. The fact that there is very little overlapping of the ranges of liquid limit values for these two horizons seems to indicate that the horizons are significantly different in regard to this property.

Table 1. Summary of Statistical Data on Index Properties

Col. No. 1	2	3	4	5	6	7	8	9	10	11
Index Property	Soil Type	Horizon	Min. Value	Max. Value	Mean	Standard Deviation	Coefficient of Variation	Standard Error No. Samples 3	5	10
			%	%	%	%	%	%	%	%
Liquid Limit	Birkbeck silt loam	A	30.1	43.5	37.4	5.2	13.9	3.0	2.4	1.7
		B	32.3	53.4	40.6	6.4	15.8	3.9	2.9	2.0
	Catlin silt loam	A	33.5	49.4	41.3	4.7	11.4	2.8	2.1	1.5
		B	36.5	51.3	44.8	5.2	11.6	3.1	2.3	1.7
	Drummer silty clay loam	A	45.8	60.9	54.6	3.8	7.0	2.2	1.7	1.2
		B	43.7	60.2	52.2	5.5	10.5	3.3	2.5	1.7
	Flanagan silt loam	A	35.7	50.1	42.9	4.1	9.6	2.4	1.8	1.3
		B	46.5	58.8	51.3	3.9	7.6	2.2	1.7	1.2
Plasticity Index	Birkbeck silt loam	A	6.8	17.2	11.9	3.8	31.9	2.4	1.7	1.2
		B	9.4	26.4	16.2	5.5	34.0	3.3	2.5	1.7
	Catlin silt loam	A	8.8	20.5	13.9	3.6	25.9	2.2	1.6	1.1
		B	10.8	23.4	18.9	4.7	24.9	2.7	2.1	1.5
	Drummer silty clay loam	A	20.5	31.3	26.1	3.5	13.4	2.1	1.6	1.1
		B	18.7	35.5	27.9	5.2	18.6	2.9	2.3	1.6
	Flanagan silt loam	A	12.2	20.9	16.6	2.7	16.3	1.6	1.2	0.9
		B	18.8	30.8	25.1	3.9	15.5	2.3	1.8	1.2
Clay Content	Birkbeck silt loam	A	17.4	28.9	22.3	4.2	18.8	2.5	1.9	1.3
		B	23.0	39.2	31.7	5.1	16.1	3.0	2.3	1.6
	Catlin silt loam	A	17.4	30.7	24.4	3.5	14.3	2.1	1.6	1.1
		B	21.2	35.9	30.8	4.7	15.3	2.8	2.1	1.5
	Drummer silty clay loam	A	35.3	42.0	37.3	1.9	5.1	1.1	0.9	0.6
		B	30.8	43.7	37.2	3.6	9.7	2.1	1.6	1.2
	Flanagan silt loam	A	22.5	29.4	25.9	1.8	6.9	1.1	0.8	0.6
		B	31.3	41.5	35.8	3.2	8.9	1.8	1.4	1.0

Table 1 shows that the values of the standard deviation for all three index properties are of the same order of magnitude. However, the liquid limits show slightly larger deviations than the plasticity indexes, while the deviations of the clay content are generally the smallest. The range in variation of the properties of each horizon is shown more clearly in Table 2, where the values in Cols. 3 through 8 represent the mean values of the index property minus or plus 2.26 standard deviations. These values provide the best available estimate of the range within which 95% of all sample values obtained from these soil horizons in DeWitt County will fall. Although there is obviously a great deal of overlapping of the ranges for the different horizons there are also some apparent differences.

Sometimes for the purpose of comparing the degree of variation within sampling groups with respect to different properties it is more convenient to express the range indicated by the standard deviation as a percentage of the mean value. This quantity is called the coefficient of variation. Its values are given in Col. 8 of Table 1. Obviously, these values are not all of the same order of magnitude. For any given horizon, the values for the plasticity index are considerably greater than those for either the liquid limit or clay content. The coefficient of variation not only indicates the variability of different soil properties but is also useful in comparing the amounts of variation to be expected in different soil horizons. Columns 9, 10, and 11 of Table 2 compare the variability of each soil horizon on the basis of the relative magnitudes of the coefficients of variation for the three properties. These rankings indicate that Birkbeck silt loam is the most variable type, Catlin silt loam the next, and Flanagan silt loam and Drummer silty clay loam the least.

The standard deviation and the coefficient of variation provide estimates of the variations to be expected in determining a given index property of a soil horizon. They do not of themselves solve the problem of selecting the number of samples required from each horizon to determine the mean index value of

Table 2. Variability of Soil Horizons

Col. No. 1	2	3	4	5	6	7	8	9	10	11
Soil Type	Horizon	Range for 95 % of Samples*						Rank in Variation**		
		LL %		PI %		Clay %		LL	PI	Clay
Birkbeck	A	25.7	49.1	3.3	20.5	12.9	31.7	2	2	1
silt loam	B	26.1	55.0	3.8	28.6	20.2	43.2	1	1	2
Catlin	A	30.7	51.9	5.8	22.0	16.5	32.3	4	3	4
silt loam	B	33.1	56.6	8.3	29.5	20.2	41.4	3	4	3
Drummer	A	46.0	63.2	18.1	34.0	33.0	41.6	8	8	8
silty clay loam	B	39.8	64.6	16.0	39.7	29.1	45.3	5	5	5
Flanagan	A	33.5	52.1	10.5	22.7	21.7	30.0	6	6	7
silt loam	B	42.5	60.1	16.3	33.9	28.8	43.0	7	7	6

* Mean ± 2.26 standard deviations

** As measured by coefficient of variation, No. 1 most variable, No. 8 least variable.

Table 3. Number of Samples for Given Accuracy

Soil Type	Horizon	Number of Samples Required*		
		LL ± 5%	PI ± 5%	Clay ± 5%
Birkbeck	A	6	4	4
silt loam	B	9	7	6
Catlin	A	5	3	3
Silt loam	B	6	5	5
Drummer	A	3	3	1
silty clay loam	B	6	6	3
Flanagan	A	4	2	1
silt loam	B	4	4	3

* For estimating range of population mean at a limit of accuracy, 95 %.

the population with a given degree of precision. This problem can best be solved by the use of two other statistical parameters, the standard error of the mean and the limit of accuracy. The mean of the samples plus or minus the standard error based on the population defines a range within which the population mean of the property will be found 68% of the time. The standard error of the mean based on 10 samples is the best available estimate of the standard error based on the population. Thus, Cols. 6 and 11, Table 1 indicate that the population mean liquid limit of the A-horizons in the Birkbeck silt loam in DeWitt County will fall 68% of the time in the range 37.4 ± 1.7% or 35.7 to 39.1%. Since this statement is based on the standard error of 10 items, the range will actually be exceeded somewhat more than 68% of the time.

The value of the standard error varies inversely with the square root of the number of samples tested if the standard deviation is considered constant. Thus, it may be used to estimate the range within which the population mean would be found for any given number of samples. For example, if the mean value of liquid limit had been obtained from tests on 3 samples instead of ten, it could be stated that the chances are about 65 out of 100 that the population mean liquid limit for the Birkbeck A horizon falls within the range 34.4 to 40.4% (Col. 9, Table 1). A comparison of the values given in Cols. 9, 10, and 11 indicates that they are approximately of the same order of magnitude but that there is an important increase in precision obtained by increasing the number of samples from 3 to 5 to 10.

It is often desirable to establish the mean of the population with a degree of precision greater than 68%. By using the standard error and values taken from the "t" distribution it is possible to determine with any desired degree of precision the range within which the population mean should fall. The "t" distribution is similar to a normal distribution and approaches it closely if it is based on a very large number of samples. However, with a small number of samples the "t" distribution is flattened and spread out so that to include

any given per cent of the area under the curve the interval must be longer than that obtained under a normal curve. The "t" value can be selected from prepared statistical tables on the basis of the number of samples and the desired degree of precision. If the value of "t" at the 95% point is multiplied by the standard error of the mean the product is the limit of accuracy, 95%. The sample mean plus or minus the limit of accuracy, 95% provides a range known as the 95% confidence limits. The value of "t" for 10 samples is 2.26, the mean liquid limit of Birkbeck silt loam is 37.4% and the standard error is 1.7%. Thus, the limit of accuracy, 95% is 1.7 x 2.26 = 3.8% and the 95% confidence limits are 37.4 ± 3.8%.

The variation of the limit of accuracy with the number of samples is shown graphically in Figs. 1, 2, and 3. By the use of these curves it is possible to obtain a solution to the sampling problem. For example, it may be desirable to determine the number of samples of each of the soil horizons required to establish with 95% confidence the population mean liquid limit value plus or minus 5%. Fig. 1A indicates the need for 3 samples of Drummer, 4 samples of Flanagan, 5 samples of Catlin and 6 samples of Birkbeck in order to meet the criteria for the A horizon. Similar data may be obtained from Fig. 1B. Data on the mean plasticity index and clay content values may be picked off of the curves in Figs. 2 and 3. These values are summarized in Table 3 for the arbitrarily selected range of plus or minus 5%. This table shows that there is a considerable variation in the number of samples required to establish the various means with the same degree of precision and confidence. Comparison of these data with those in Cols. 9-11, Table 2 indicates the influence of variability in the basic data upon the number of samples required to establish the means of the population within the specified limits of accuracy. Nearly twice as many samples are required to establish the mean index properties of a variable soil type such as Birkbeck silt loam with the same degree of precision as are required to characterize a relatively uniform type such as Flanagan silt loam. Figs. 1, 2, and 3 may also be used to determine for any specified number of samples from 1 to 20, the 95% confidence limits for the population means of each of the three properties.

From the strict statistical viewpoint it is not possible to apply the results of the investigations of one soil type in one county to other soil types in the same or other counties. Nevertheless, the information contained in Figs. 1 to 3 can be used as a general guide in planning sampling operations, especially for loess-derived soils such as the ones included in this study in DeWitt County. When a similar study was begun to classify the pedologic soil types in Livingston County, Illinois, these data formed the basis for selecting 5 profiles of each soil type for sampling and testing.(7) In taking this number of samples it may be anticipated that the population mean index values of variable soil types such as Birkbeck silt loam will be established (at a limit of accuracy, 95%) about in the following ranges:

Sample mean liquid limit	± 6.5%,
Sample mean plasticity index	± 5.5%,
Sample mean clay content	± 5.0%.

On the other hand the same values for relatively uniform soil types such as Flanagan silt loam would be expected to fall in these ranges:

Sample mean liquid limit	± 4.0%,
Sample mean plasticity index	± 4.0%,
Sample mean clay content	± 3.0%.

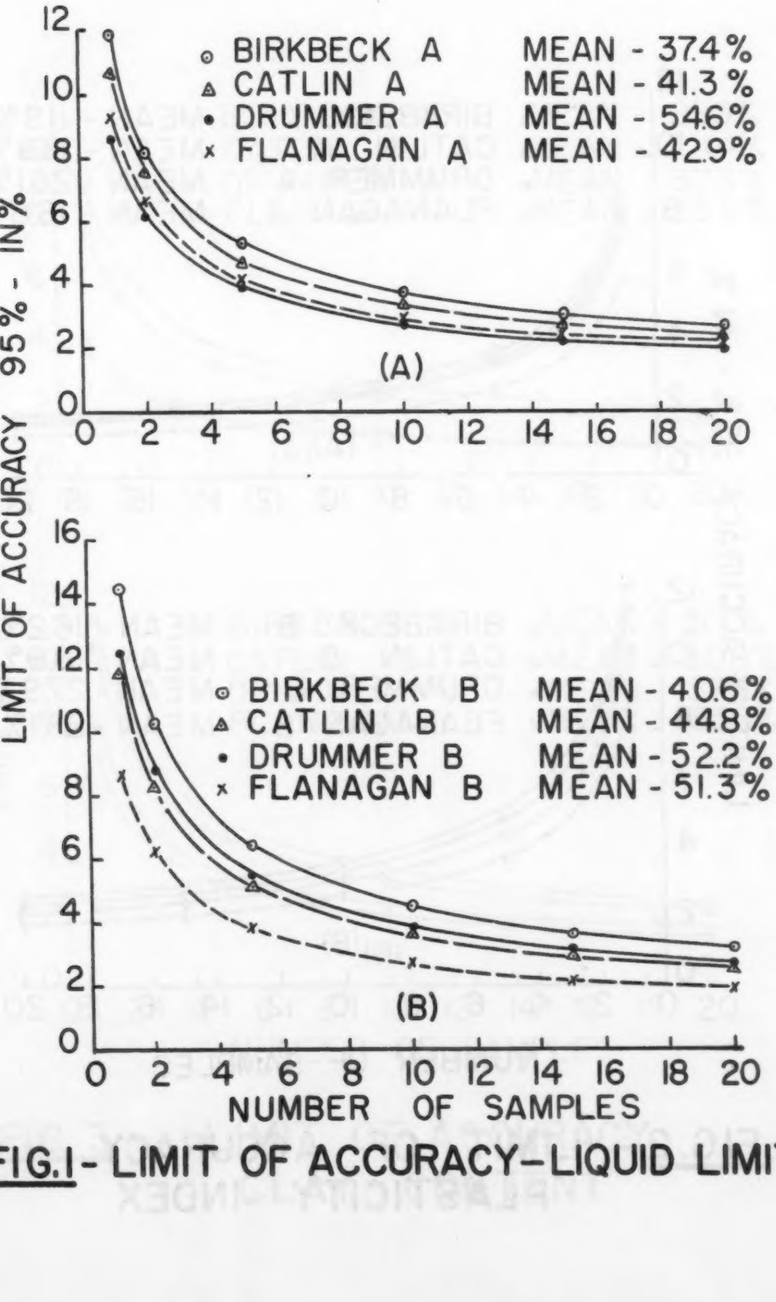

FIG.1 - LIMIT OF ACCURACY-LIQUID LIMIT

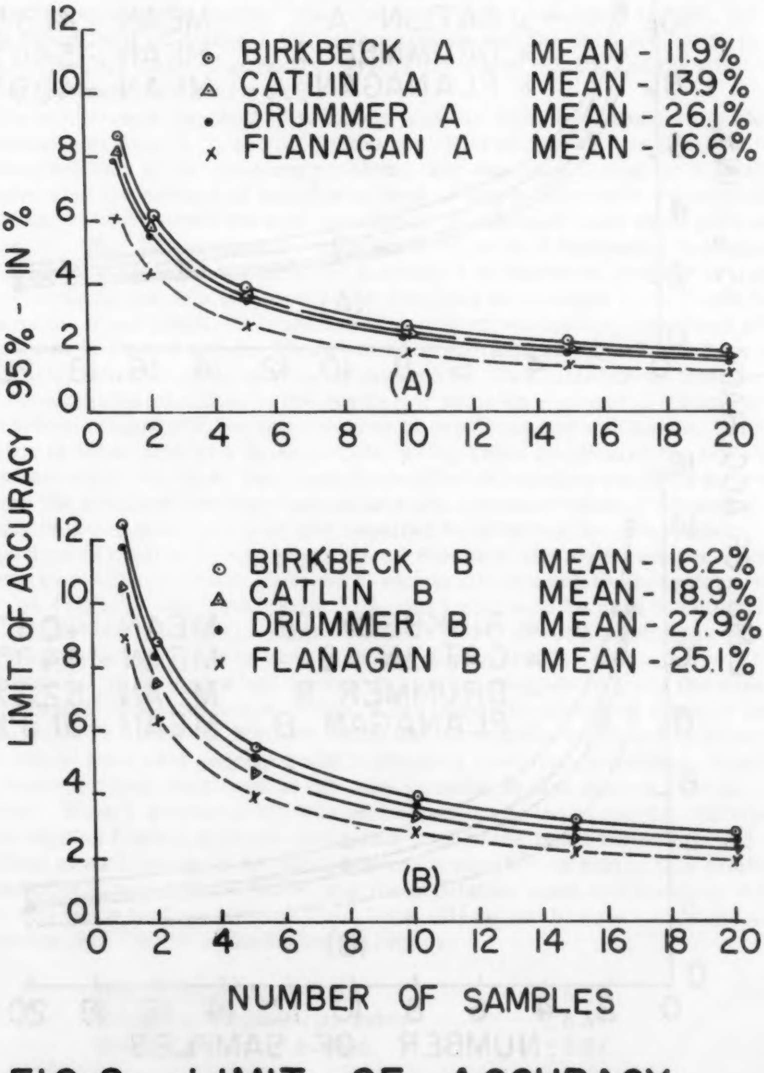

FIG. 2 - LIMIT OF ACCURACY PLASTICITY INDEX

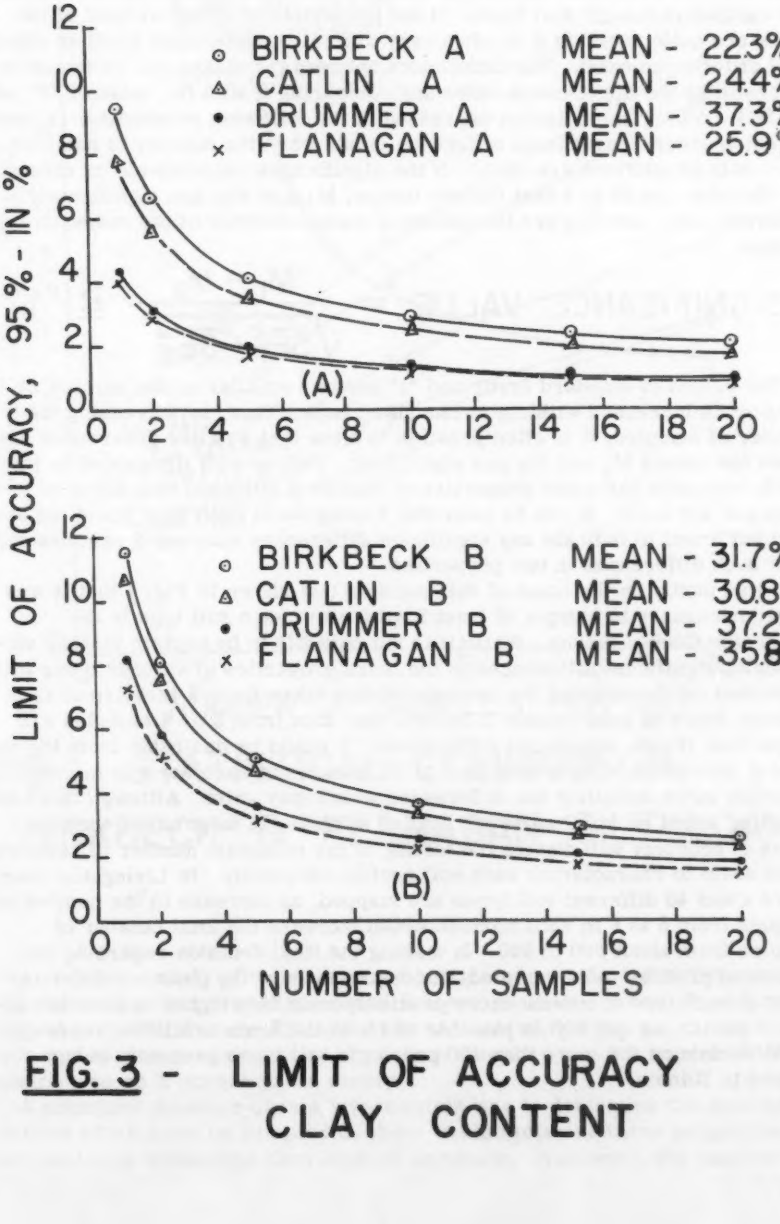

FIG. 3 - LIMIT OF ACCURACY CLAY CONTENT

Although the degree of validity of these estimates in a new sampling area cannot be stated, they provide a guide for determining the relative accuracy of any proposed program.

The statistical information obtained from this analysis has other uses. One of the aims of the cooperative investigation, Soil Exploration and Mapping, is to determine what differences may exist between the physical properties of the various pedologic soil types. If the properties of either or both of two types are quite variable it is often very difficult to determine whether significant differences exist. The differences between the means can be tested by determining the significance value and comparing it with the value of "t", at the 95% point. The criterion expressed in the following relationship is used to test whether a significant difference exists when the number of samples from both populations are equal. If the significance value equals or exceeds "t" the odds are 19 to 1 that the two means, M_1 and M_2, are significantly different. SE_1 and SE_2 are the values of standard error of the respective means.

$$\text{SIGNIFICANCE VALUE} = \frac{M_1 - M_2}{\sqrt{\overline{SE}_1^2 + \overline{SE}_2^2}} \geqq \text{"t"}$$

The values of standard error and "t" become smaller as the number of samples is increased within a reasonable range. Thus, by increasing the number of samples, it is often possible to show that smaller differences between the means M_1 and M_2 are significant. This is well illustrated in Fig. 4 which compares the index properties of Birkbeck silt loam with those of Flanagan silt loam. It can be seen that 3 samples of each type would not have been sufficient to indicate any significant differences whereas 5 samples did show such differences in two properties.

It was partly on the basis of data such as that shown in Fig. 4 that it was judged necessary to sample at least 5 profiles of each soil type in the Livingston County survey. While it is not possible to be certain that all statistically significant differences in the index properties of various types will be evident on the basis of the analysis of data taken from 5 samples of each horizon, there is good reason to believe that data from 3 or 4 samples will reveal few, if any, significant differences. It would be desirable from the statistical standpoint to have data on 8 to 10 samples of each horizon in order to establish more definitely the differences which may exist. Although this added sampling might be worthwhile for limited surveys, in most cases considerations of economy will dictate the testing of the minimum number of samples which seem to characterize each soil profile adequately. In Livingston County, where about 40 different soil types are mapped, an increase in the number of samples from 5 to 8 in each horizon would increase the total number of samples from about 600 to 960. In making the final decision regarding the number of profiles to be sampled, the desire to know the physical characteristics of each type somewhat more precisely must be weighed against the desire to gather, as quickly as possible and with the funds available, representative data on the more than 400 pedologic soil types presently being mapped in Illinois.

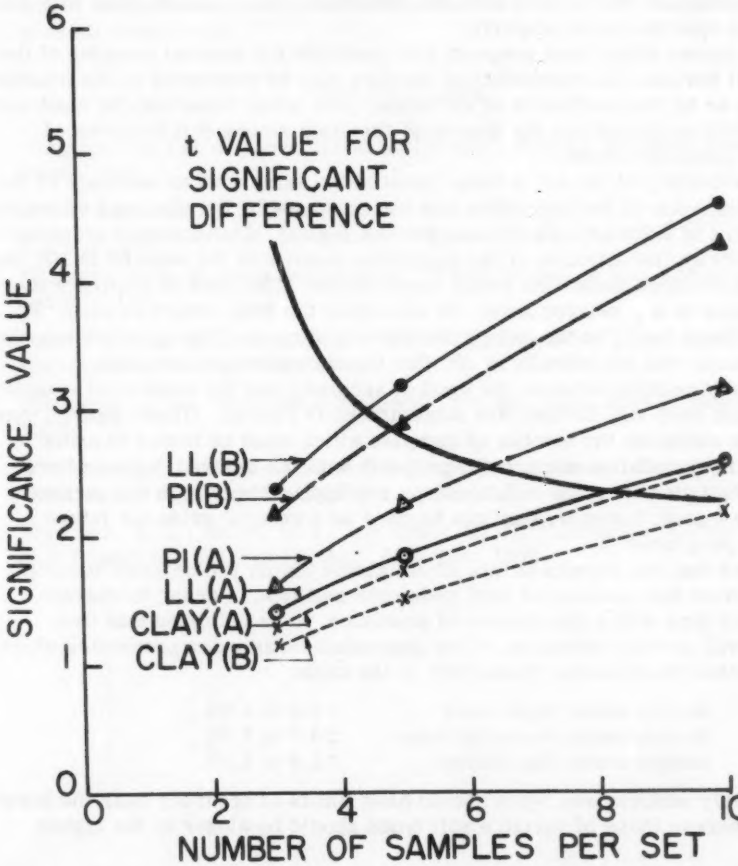

FIG.4 - STATISTICAL COMPARISON BIRKBECK & FLANAGAN SILT LOAMS

SUMMARY

A statistical study of three index properties was conducted utilizing test data obtained on samples of the A and B horizons of four pedologic soil types mapped in DeWitt County, Illinois. It is considered that the ten samples of each soil horizon were obtained by random sampling and are representative of each type as it occurs in the county.

A principal objective of this investigation was to determine the number of profiles which must be sampled in order to characterize three properties of each soil type within specified limit of accuracy. In general, the number of

samples required will depend upon the variability of the soil horizon with respect to a specific index property.

When values of an index property are available for several samples of the same soil horizon, the variability of the data may be measured by the standard deviation or by the coefficient of variation. The latter value may be used more conveniently in comparing the degree of variation among soil horizons of different pedologic types.

The reliability of the mean index value of the sample as an estimate of the mean index value of the population can be expressed by the standard error or by the limit of accuracy, p% (p% confidence limits). The standard error defines limits on the estimate of the population mean with the odds 68 in 100 that the true population mean lies within these limits. The limit of accuracy may be computed at any desired level. In this study the 95% level was used. By applying these limits to the sample means an estimate of the population means may be made with the odds 95 in 100 that the estimates are accurate.

The relationships between the limit of accuracy and the number of samples tested from each soil horizon are summarized in figures. These figures may be used to establish the number of samples which must be tested in order to estimate the population mean index property with the desired degree of precision. Statistically, these relationships are applicable only to the particular soil types tested; however, they can be used as a general guide for future sampling programs.

It is felt that the results of this investigation justify the decision to obtain samples from five profiles of each pedologic soil type in order to characterize each type with a fair degree of precision. It is indicated that five samples will provide estimates of the population mean index properties which will be within the following limits 95% of the time:

Sample mean liquid limit	± 4.0 to 6.5%,
Sample mean plasticity index	± 4.0 to 5.5%,
Sample mean clay content	± 3.0 to 5.0%.

Relatively uniform soil types should have limits of accuracy near the lower values, whereas those of variable soil types should be closer to the higher values.

The statistical analysis may also be used to test for significant differences between the properties of two pedologic types. A graph is presented to show the relationship between the significance values and the number of samples used. It appears that data on five samples of each horizon are sufficient to reveal some significant differences in index properties even between pedologic soil types derived from identical parent materials.

ACKNOWLEDGMENTS

This report was prepared as part of the Illinois Cooperative Highway Research Program, Project IHR-12, "Soil Exploration and Mapping". This investigation is conducted by the staff of the Department of Civil Engineering in the Engineering Experiment Station, University of Illinois, under the joint sponsorship of the Illinois Division of Highways and the U. S. Department of Commerce, Bureau of Public Roads. The authors are indebted to the members of the project advisory committee for suggestions regarding the form of this report. Particular acknowledgement is made to Dr. Walter C. Jacob,

Professor of Agronomy, University of Illinois, for his helpful suggestions on the statistical analysis.

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Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONSTRUCTION MATERIALS CONTROL FOR THE AASHO ROAD TEST

James F. Shook,[1] M. ASCE

SUMMARY

A brief description of some of the test methods and statistical quality control procedures used during construction of the AASHO Test Road is given. Emphasis has been placed on control of compaction of the earth embankment and the granular subbase and base courses.

INTRODUCTION

The AASHO Road Test is a 22 million dollar highway research project sponsored by the American Association of State Highway Officials.(1) It is administered and directed by the Highway Research Board of the National Academy of Sciences - National Research Council. The cost is spread between the State and Territorial Highway Departments, the Bureau of Public Roads, the Automobile Manufacturers Association and the American Petroleum Institute. Support in the form of personnel to drive the test vehicles is supplied by the Department of Defense.

The project is located near Ottawa, Illinois, about 80 miles southwest of Chicago, in an area with climate and soils typical of wide areas of the country. The Illinois Division of Highways constructed the test pavements through their normal paving contract procedures. The eight-mile length of test pavements will become part of the federal interstate highway system upon completion of the tests.

The Highway Research Board was asked to undertake the project in 1955; construction got underway in 1956 and was completed late in the summer of 1958. Test traffic was started soon thereafter and will operate for two years, after which some special studies involving heavy military vehicles will be made. Final reports should be in print in 1961, but preliminary results useful to the various highway departments should be available earlier.

Note: Discussion open until March 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2211 is part of the copyrighted Journal of the Soil Mechanics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 5, October, 1959.

1. Materials Engr., AASHO Road Test, Ottawa, Ill.

The principal objective of the Road Test is to find significant relationships between pavement performance and traffic for pavements of certain designs under controlled truck traffic of certain loadings.

In the main experiment there are eight independent variables. These are axle load, axle spacing, number of load applications, surfacing type (portland cement or asphaltic concrete), concrete reinforcement, surfacing thickness, subbase thickness and base thickness. Side experiments include consideration of base type and shoulder pavement. There is also included an independent study of 16 test bridges (steel beam, reinforced concrete and prestressed concrete) located in two of the main loops.

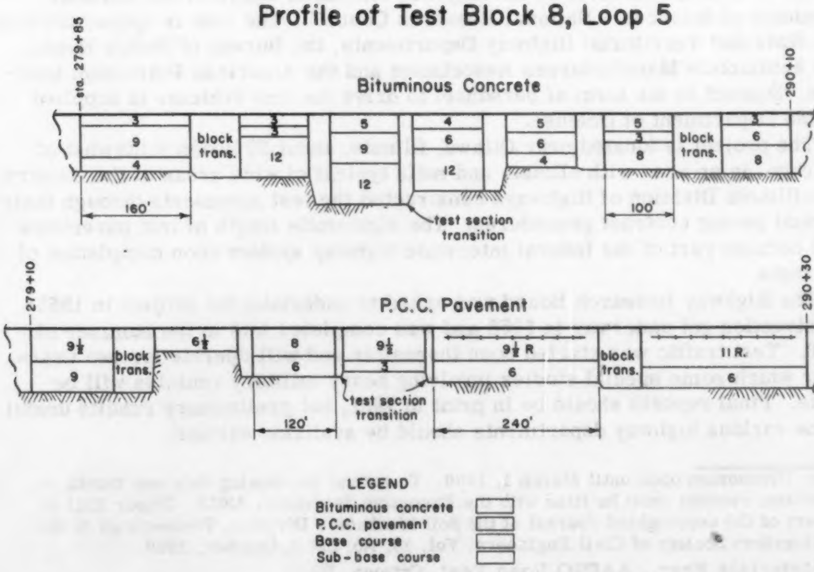
Tests under traffic are being conducted in five test loops of which four have two tangents 6800 feet long and one has two 4400-foot tangents. A sixth loop has been constructed to permit auxiliary studies on pavements not subjected to traffic.

Test loads range from 2000-lb. and 6000-lb. single axle loads on the lightest pavements to 30,000-lb. single and 48,000-lb. tandem axle loads on the thickest pavements.

The test layout consists structurally of 836 experimental units, or test sections, one lane in width and of varying lengths (100, 120, 160 and 240 feet) which have one subbase, base, and pavement thickness combination. There is a uniform 36-inch embankment of selected soil fill under all test sections. For construction purposes, these test sections were grouped into "blocks" 500 to 800 feet in length. A typical test section layout is shown in Fig. 1.

With few exceptions, types of materials and construction procedures followed standard highway practice. There was neither money nor space to include untried materials or construction techniques, and the variables included in the experiment were those considered most important by a majority of the sponsors of and advisors to the project.

Profile of Test Block 8, Loop 5



Construction Materials Control

Since the main Road Test experiment involved only one soil, subbase or base type uniformity of all materials over the entire project was essential. To insure this uniformity it was necessary to plan and carry out an extensive program for materials and construction control.

Throughout the planning and execution of the program for materials control modern experiment design principles were followed and statistical quality control techniques were utilized wherever possible. This program required the taking of many tests during all phases of construction, yet it was imperative that the contractor be allowed to proceed with his work as rapidly as possible. As a result several special testing methods were developed and used, assembly line techniques were adopted and communication was speeded by the use of mobile radio units in all field vehicles assigned to testing crews.

Construction materials control was the joint responsibility of the Illinois Division of Highways and the Highway Research Board laboratory on the project. Acceptance tests of materials off the site were made by the Illinois Division of Highways. On-site testing was largely the responsibility of the materials laboratory of the Road Test.

This paper is concerned primarily with the work of the Road Test materials laboratory, the procedures used in controlling construction of the earth embankment, granular subbase and base components of the roadway.

Compaction Control, Earth Embankment

The entire test road was constructed on a 3-ft. embankment of selected soil. This material was obtained on the job site from three borrow pits of uniform quality, representing typical low quality soils available in many areas in this country. The soil used was a yellow-brown, A-6 soil having a group index between 9 and 13. Characteristics included, from typical test data, a liquid limit of 30, a plasticity index of 14, 80 per cent passing the No. 200 sieve, and 40 per cent finer than 0.005 mm.

Specifications were set up requiring compaction to between 95 and 100 per cent of standard AASHO density at optimum moisture content plus or minus 2 percentage points. However, to assist in securing uniformity of compaction, an attempt was made to keep the moisture content near the high side of the specified limits, slightly above optimum moisture content.

Altogether, over 7500 field density tests and 4000 maximum density tests were made during the construction. The job average per cent compaction was very close to 97.5 at approximately 1 per cent above optimum moisture, and the desired uniformity was obtained.

The earth embankment was constructed in 4-inch compacted layers one full construction block in length. Water was added and the soil pulverized by teams of three to five rotary speed mixers. Compaction was accomplished with pneumatic-tired super-compactors having a 15-ton total weight, operating in a rigidly controlled rolling pattern.

Tight control of water was obtained by having trained technicians follow each team of rotary speed mixers to set, by "feel", the rate at which water was applied to the soil. All water was added through spray bars spraying directly on the soil as it was pulverized, and it was metered from tank wagons attached to the mixers. Moisture content was not further checked until density tests were made.

For testing purposes, each 500 to 800-foot construction block was divided into sections corresponding in length to the 100, 120, 160 or one-half of the 240-foot test sections (Fig. 1). Two field density tests and one maximum density test were made in each section, making a total of four to twelve density tests in each block. To obtain per cent compaction, the individual field densities were divided by the block average maximum density.

The decision to accept or reject a block-lift was based on a statistical analysis of the compaction data obtained for that lift. In order for any statistical analysis of the compaction data to be valid, it was first necessary for the data to have been obtained from samples which were without bias; hence the locations for all tests within each section were selected from a table of random numbers. It was not left up to the inspector to select "average" or "good" or "bad" spots for his tests.

The details of the analysis procedure are too involved for discussion in this paper; however, a brief description will be given. Fig. 2 shows a sample computation sheet and Appendix A gives some explanation of the terms used. The per cents compaction obtained from one block lift were recorded and the mean and standard deviation computed. From these computations the per cent of tests which would have been out of the specified limits (95 to 100 per cent compaction) had a very large number of tests been made was estimated ($P_U + P_L$ in Fig. 2) and used for accepting or rejecting the work. An acceptable percentage out was selected after the job had been running for a short while. If the estimated percentage out of specifications was greater than this value the particular block-lift involved was reworked as necessary.

This objective system for acceptance or rejection on the basis of statistical analysis was considered highly effective in that through unbiased decisions it helped to insure uniformity of construction and at the same time recognized the variability inherent in construction and testing techniques. There were as many as four resident engineers in charge of construction at one time. To allow each to make his own independent decision would have left much to be desired from the standpoint of overall job uniformity. To require that all tests fall within the specified range in per cent compaction was considered too restrictive; and to require only the average to be within the band would have been unnecessarily lenient. The procedure mentioned here, in effect, controlled the average or mean density of each construction block and the variability within each block, yet, by allowing a reasonable estimated percentage out of the specifications ($P_U + P_L$), gave consideration to normal construction and testing variability.

Acceptance in most cases was based on an allowable 45 percentage out for individual block-lifts—top lifts were held to 40 per cent. However, for the job as a whole the average block-lift percentage out was about 20 per cent.

Up to this point, mention has been made of the number of tests required, the frequency of tests and the analysis used in constructing the earth embankment for the Road Test. Some discussion of the testing procedures themselves is now in order.

At one time there were four five-man mobile field crews, 25 laboratory technicians and other help, making a total of over 50 engineers and technicians doing construction control for the materials laboratory alone. Whenever possible, they were set up for assembly-line production. In most cases, the contractor had his acceptance of a block-lift in one and one-half hours after completion of the rolling operation. As many as 25 block-lifts were completed in one day.

Figure 2

COMPACTION CONTROL DATA ANALYSIS

TANGENT 6-1-7 LIFT 10 STATION ________ TO ________

STONE BASE ☐ GRAVEL BASE ☐ SOIL ☒

CEMENT TREATED BASE ☐ SUBBASE ☐ DATE 10/17/56

STATION	COORD. W-E	COORD. ℄	% COMP (X)	STATION	COORD. W-E	COORD. ℄	% COMP (X)
			97.3				
			99.2				
			99.0				
			101.0				
			98.5				
			97.2				

Total Number Samples, N = 6

Grand Total, Σx = 592.2

Grand Mean, $\Sigma x / N = \bar{x}$ = 98.7

Σx^2 = 59460.02

$(\Sigma x)^2 / N$ = 58450.14

SS (x) = 9.88

SS (x) / (N − 1) = s^2 = 1.976

s = 1.41

No. Samp. Out

Upper Spec. U = 100 — 1

Lower Spec L = 95 — 0

$C_U = (U - \bar{x}) / S$ = 1.3/1.4 = 0.92

Est % Above U = P_U = 18.6

$C_L = (\bar{x} - L) / S$ = 3.7/1.40 = 2.64

Est % Below L = P_L = 0.0

Total Est % Outside Spec $P_U + P_L$ = 18.6

Remarks: ________

Action: Accepted

AASHO No. 158 1-31-58 1K

One of the greatest sources of delay in soil compaction control is that of determining the moisture content of the samples. On the AASHO Road Test, a continuous drying oven was built which contributed as much as any single item to the speed of testing. The main features are shown in Fig. 3. It consisted basically of an endless chain which passed the weighed wet samples of soil under a battery of infra-red heat lamps. With the use of tared and numbered ointment boxes and scales at both ends of the oven, samples could be kept drying continuously. One sample took 23 minutes to dry; and samples could be dried at the rate of four per minute. As many as 1200 samples were dried through this oven in one day; and altogether over 70,000 samples have passed through the dryer in the past two years.

Maximum densities were determined by a procedure which made use of one molded specimen, the proctor penetration needle and a family of moisture-density-proctor needle reading curves prepared from standard tests on the Road Test soil. Fig. 4 shows a section of the family of curves and indicates

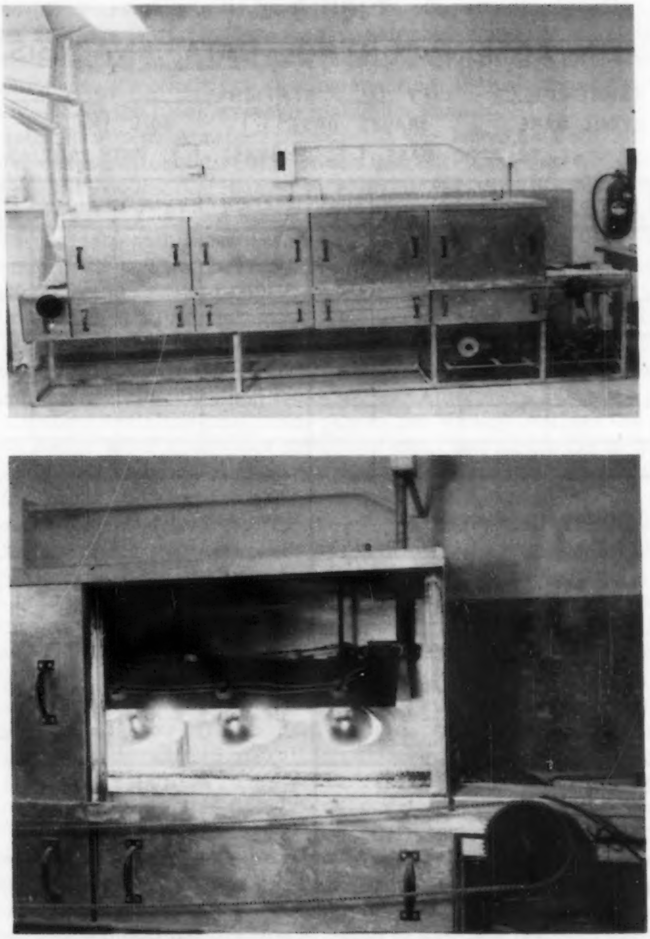


Figure 3 Continuous Drying Oven

how they were used. Wet densities corresponding to optimum conditions were read directly from the curve, but optimum moistures were obtained by adjusting the moisture contents of the molded specimens as indicated in Fig. 4—the illustrative problem shows a sample moisture content of 14.8 adjusted plus 0.4 percentage points to 15.2 for an optimum. Wet densities were converted to dry densities before per cents compaction were computed.

Samples of soil for the maximum density tests were obtained from selected spots on the road before the water was added with the rotary mixers and brought to the central laboratory for processing. In the laboratory, several men pulverized the soil, sieved out the plus 1/4-inch material and passed the sample to another group which blended water with the sample, by "feel", to within a few percentage points of optimum. Another crew compacted one standard mold (4-inch diameter mold, 5.5 lb. hammer, 12-inch drop), weighed, obtained the penetration needle reading and removed a moisture sample from

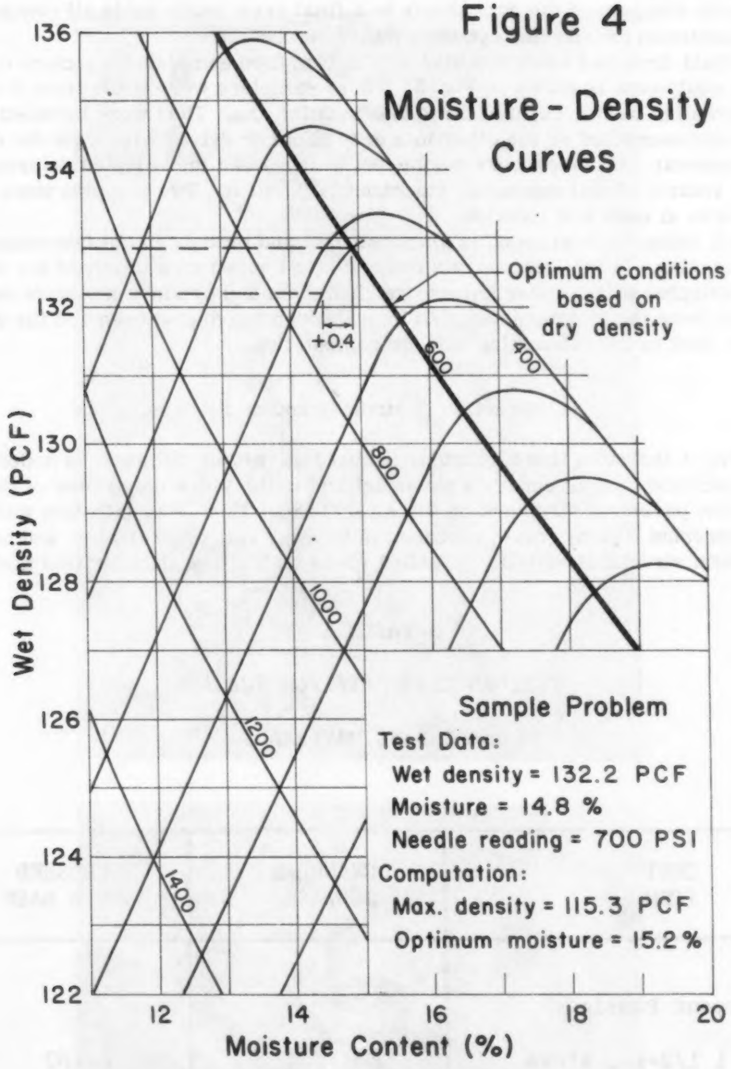


Figure 4
Moisture - Density Curves
Optimum conditions based on dry density
+0.4
400
600
800
1000
1200
1400
Wet Density (PCF)
136
134
132
130
128
126
124
122
12
14
16
18
20
Moisture Content (%)
Sample Problem
Test Data:
Wet density = 132.2 PCF
Moisture = 14.8 %
Needle reading = 700 PSI
Computation:
Max. density = 115.3 PCF
Optimum moisture = 15.2 %

the molded soil sample. Moisture samples were handled by a separate group of men who passed the data sheets to a final crew which made all computations of maximum density and optimum moisture content.

Field densities were obtained with driven tube samples. A picture of the test equipment is shown in Fig. 5. These samplers were made from 3-inch thin-walled tubing, cut into three 7/8-inch lengths. They were bevelled on one end and connected on the other to a drop hammer driver with a pin for ease of removal. All tubes were numbered, weighed and the volumes determined. The volume of soil measured was about 1/70 cu. ft. Two samples were usually taken at each test location.

All tubes were wrapped in aluminum foil and brought to the laboratory for processing. In the laboratory a crew of about seven men trimmed the ends of the samples with butcher knives, weighed them and obtained moisture samples. From here the moisture samples went through the drying oven and the data were sent to the calculating unit for computation.

Compaction Control, Granular Subbase

Fig. 1 indicates that a granular subbase of varying thicknesses underlays the portland cement concrete pavement and is the lower component of the flexible pavement structure on the AASHO Road Test. The intention was that it represent typical gravel subbases in highway use, be drainable, and have medium structural stability. Table 1 gives typical test data for this material.

TABLE 1

TYPICAL TEST DATA FOR SUBBASE

AND BASE MATERIALS

TEST ITEM	GRANULAR SUBBASE	CRUSHED STONE BASE
Percent Passing		
1 1/2-in. sieve		100
1-in. sieve	100	90
1/2-in. sieve	91	68
No. 4 sieve	75	50
No. 40 sieve	27	20
No. 200 sieve	7	11
Plasticity	NP	NP

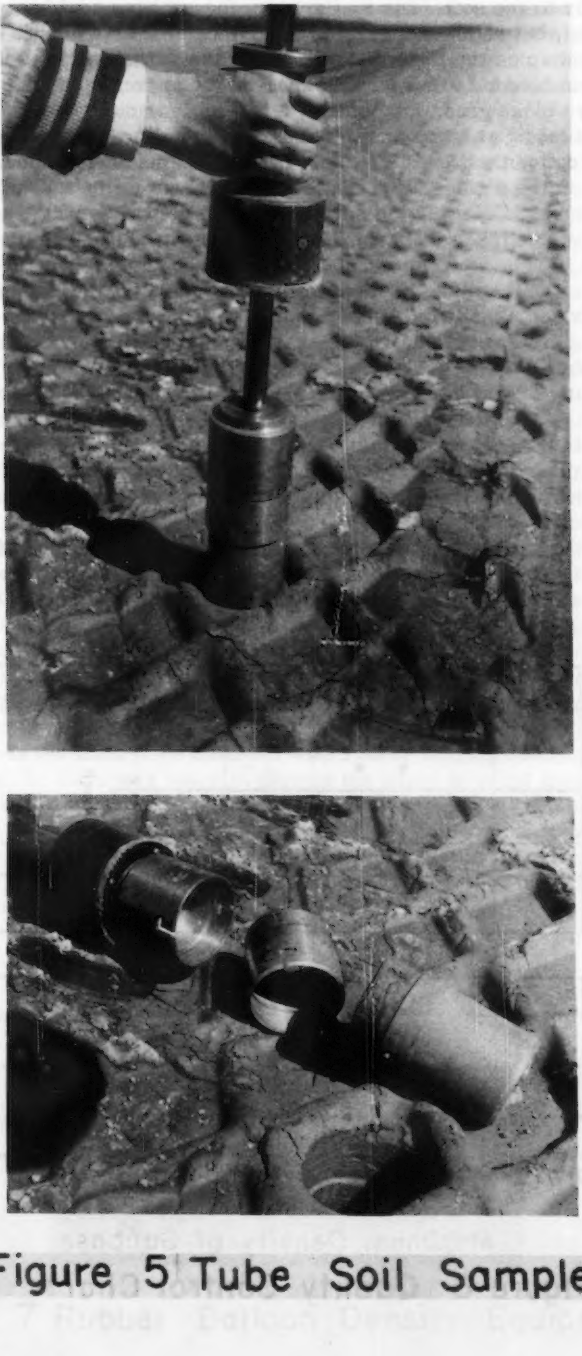


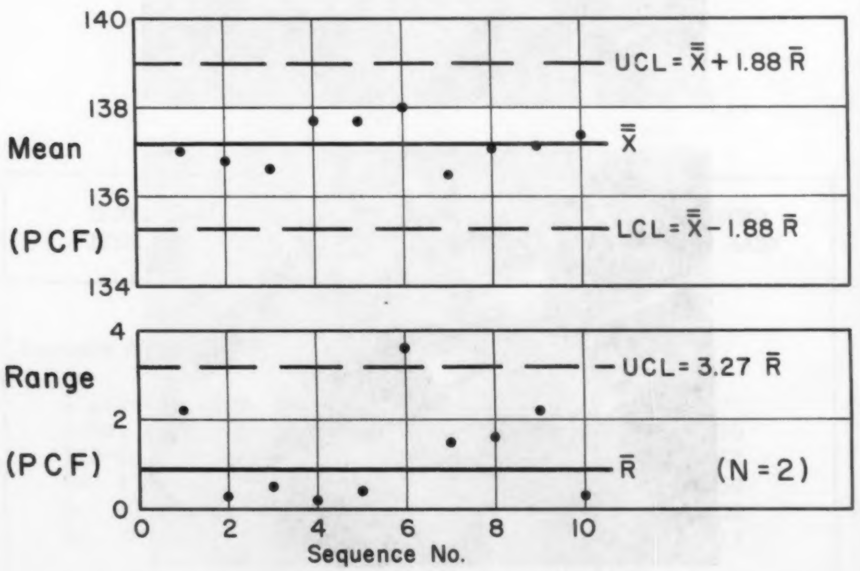
Figure 5 Tube Soil Sampler

The considerations of uniformity over the test road applied to this material as well as to the soil. For that reason, the subbase could not be a bank-run material, but rather material produced under a high degree of control. A plant was set up on the test road site where the raw material was washed, screened and blended to meet rigid specifications.

To insure close gradation control, a job-mix formula was set up with limits of plus or minus 5 per cent on sieves larger than No. 10, 3 per cent on No. 40, and 2 per cent on the No. 200 sieve. Tests were made continuously throughout the day at the plant. Six gradation samples were also taken from each block on the roadway after the spreading operation. Over 99 per cent of more than 700 samples taken from the roadway were within the specified tolerance limits.

In addition to the direct comparison to a gradation requirement, results from the six tests made on samples obtained from the road were plotted on standard quality control charts for analysis. A detailed description of this procedure cannot be given here; however, Fig. 6 shows the same type of chart used for study of maximum densities. (For a discussion of control charts see reference 3.) The gradation control chart showed means and ranges for subgroups of six (N = 6), for certain selected sieves. Very few of the means were out of control, indicating that for the job as a whole gradation control was excellent when compared to the variation among the six tests in one construction block.

Most of the subbase material was placed on the road in loose layers from 4 to 11 inches thick with self-propelled mechanical spreaders loaded from the shoulder area, given nominal compaction, and left as mulch cover over the



Maximum Density of Subbase

Figure 6 Quality Control Chart

winter of 1957-58. The following summer this material was compacted to the specified density and new material added where needed. Material in place was scarified, sprinkled with water from trucks operating on the shoulder and compacted with several passes of pad-type vibratory compactors. New material was generally placed at or near the proper moisture content and was not scarified.

The specifications required that the subbase material be compacted to between 100 and 105 per cent of maximum laboratory density at plus or minus one percentage point from an optimum moisture content determined in the field. The maximum density was determined using the standard AASHO procedure except that all plus 1/2-inch material was removed and replaced with 1/2-inch to No. 4 material. Full moisture-density curves were made on composite samples of subbase material taken from the roadway prior to compaction.

Full use was made of the type of quality control chart shown in Fig. 6. (For a discussion of control charts see reference 3.) Maximum densities from all tests were grouped in sets of two as they were completed and the means and ranges plotted as shown. The average range for each loop was computed and control limits for means calculated. If all means of two were within the control limits, the various tests were considered to be of the same universe of densities and the overall mean was used as the maximum density to which field densities were compared.

Field densities were taken by mobile field crews at randomly selected locations. However, because of the variations in thickness among the sections (see Fig. 1) it was not practicable to stay with the block-lift plan used for the soil. Each lift was considered as before, but only one or two sections could be accepted or rejected at a time. Therefore, from four to six tests were set as a minimum for rejecting the work at any one place. The analysis of the per cents compaction followed the procedure previously mentioned for the soil (see Fig. 2).

Densities in the field were obtained with rubber balloon devices of the type shown in Fig. 7. Care was taken in digging the holes to avoid loosening the granular material along the sides of the hole. The material removed from the hole was sent to the central laboratory, in friction-top cans, where it was weighed, the moisture content determined, and all calculations were made. Again, radio communication and the continuous drying oven played an important part in speeding the work.

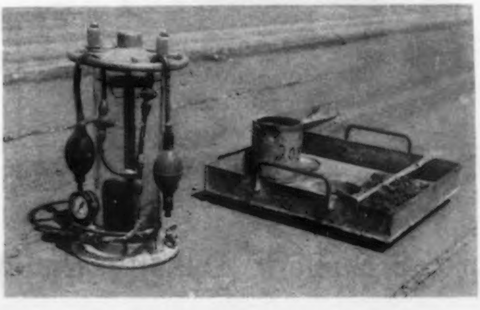


Figure 7 Rubber Balloon Density Equipment

Approximately 150 tests were made to determine maximum density and over 2000 field density tests were run. Each unit or group of sections was usually accepted on the basis of an allowable 35 percentage out computed from the test data as described previously for the soil, but the average unit percentage out was only about 15.

Compaction Control, Crushed Stone Base

All of the base under the flexible pavement (again see Fig. 1) was a high-quality crushed dolomitic limestone produced and placed under close supervision. Typical test data are shown in Table 1.

This material was furnished by a nearby materials producer in two sizes, plus a special No. 40 to No. 200 blending material. It was weigh-batched on the job into trucks and hauled to the road where it was mixed to the proper water content in a 34 E dual-drum concrete paving mixer. Spreading was accomplished with a self-propelled mechanical spreader which was filled from a belt operating off the front of the mixer. The desired density was obtained with a pneumatic-tired roller followed by a steel three-wheeled roller.

Unlike the subbase, the base was laid in 3-inch compacted layers. However, the requirement of compaction to between 100 and 105 per cent of maximum laboratory density, basic procedures and testing techniques, and analyses of data were essentially as described for the subbase. Only in the method for obtaining densities in the field was there a radical departure.

The method used for obtaining densities of the compacted base material made use of a nuclear density device developed and built on the project.(2) A picture of the unit in use is shown in Fig. 8. It consisted of a surface probe,

Figure 8 Nuclear Density Equipment

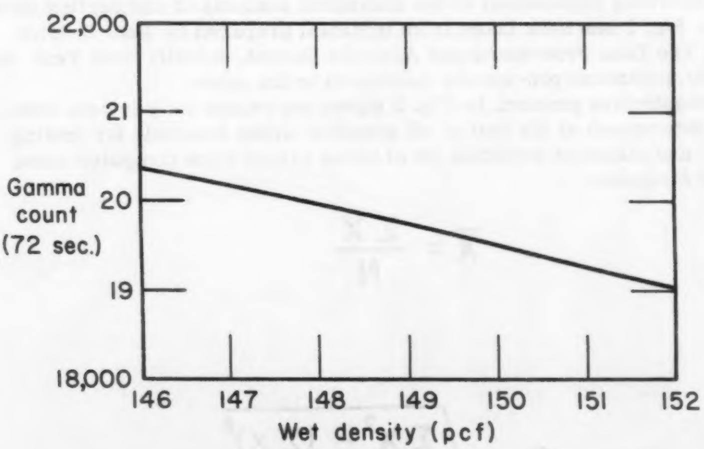
which contained a source of gamma-radiation and Geiger-Muller tubes separated by lead, an accurate timing device, and a scaler or counter. The unit was also portable and battery powered. It worked upon the principle that the amount of radiation which was picked up by the Geiger-Muller tubes was proportional to wet density and could, with proper calibration, be used to measure density. A typical calibration curve is shown in Fig. 9.

Operation was relatively simple. The probe was placed upon the compacted base course at the spot selected for test and three 72-second counts made. An area approximately 8 inches in diameter was covered as the probe was rotated between the three counts. The average of these three counts was used to estimate wet density, using the calibration curve shown in Fig. 9. A moisture sample was obtained with a shovel, placed in sealed cans and sent to the laboratory where it was dried in the continuous oven. Per cents compaction were then computed and the analysis for acceptance made.

This device was used with success on the Road Test, where it became clear early in the planning stage that the desired number of tests could not be made with conventional procedures. About 1000 tests were made altogether, and the degree of control was better than that for the subbase; the job average unit percentage out being less than 10. Several items contributed to this low variability in density, among them being the extreme care in batching, mixing and placing the base. However, it is also felt that the nuclear density device is inherently more accurate than devices which measure the volume of a hole dug into the material.

CONCLUSION

A brief description of testing procedures and statistical quality control methods used on the AASHO Road Test has been presented. More complete



Calibration Curve (Crushed Stone)

Figure 9

descriptions will be presented in Road Test reports to be issued later. At the time of writing the paper neither time nor policy would allow the presenting of detailed test data and these also will be given in subsequent reports.

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APPENDIX A

Explanation of Table 2

The following explanation of the statistical analysis of compaction data shown in Fig. 2 has been taken from material prepared by Paul E. Irick, Chief of The Data Processing and Analysis Branch, AASHO Road Test, who set up the statistical procedures mentioned in the paper.

The illustrative problem in Fig. 2 shows six values for per cent compaction determined at six (out of all possible) spots available for testing. The mean ($\bar{x}$) and standard deviation (S) of these values were computed using standard formulas:

$$\bar{X} = \frac{\Sigma X}{N}$$

$$S = \sqrt{\frac{\Sigma X^2 - \frac{(\Sigma X)^2}{N}}{N - 1}}$$

Next a calculation was made to determine how many standard deviations the mean ($\bar{X}$) was from the upper (U) and lower (L) specification limits:

$$C_U = \frac{U-\bar{X}}{S}$$

$$C_L = \frac{\bar{X}-L}{S}$$

The ratios C_U and C_L were then used to estimate the per cent of tests above (P_U) and below (P_L) the specification limits U and L which would have been found had a very large number of tests actually been made. This estimation was made from published Tables by Lieberman and Resnikoff.*

The work was accepted if $P_U + P_L$ did not exceed some arbitrarily chosen value.

Note that in the illustrative example $P_U + P_L$ was calculated to be 18.6 per cent, although actually 16.7 per cent, or one-sixth, of the individual tests were outside the specification limit.

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*Lieberman, G. J., and Resnikoff, G. J., "Sampling Plans for Inspection by Variables", Journal of the American Statistical Association, June, 1955, pages 457-516.

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SUBSURFACE EXPLORATIONS IN PERMAFROST AREAS

James R. Cass, Jr.,[1] M. ASCE

SUMMARY

Soil sampling techniques used in two subsurface investigation programs undertaken in the Arctic are described and compared. Since the methods used were only partially successful in recovering samples for field testing, recommendations are made for the development of boring procedures which should prove to be more satisfactory.

SYNOPSIS

Subsurface investigation programs were undertaken in the Arctic during 1955 and 1957. Techniques which were used for obtaining permafrost samples during the two programs are described and compared. Since the sampling methods used were only partially successful in recovering soil samples for field testing, recommendations are made for the development of boring procedures which should prove to be more satisfactory.

INTRODUCTION

Subsurface investigations for building foundation and pavement design were undertaken at the Frobisher Bay Air Base, Baffin Island, Northwest Territories, Canada, during the mid-summer of 1955 and late summer of 1957 under contract with the U. S. Army, Corps of Engineers, Eastern Ocean District. This air base is located at the head of Frobisher Bay on the southern part of Baffin Island about 200 miles south of the Arctic Circle. The mean annual

Note: Discussion open until March 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2212 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 5, October, 1959.

1. Engr., Fay, Spofford & Thorndike, Inc., Boston, Mass.

temperature is 17 degrees F, with maximum summer temperatures in the low sixties and winter temperatures as low as -40 degrees F.

Frobisher Bay is accessible only by air, except during August, September and part of October, when the bulk of supplies and equipment for the year's operation of the base are brought in by sea. Personnel, perishables, and certain supplies are flown in and out of Frobisher Bay when necessary all year round.

The investigations are described in detail for each year relating the scope of each program, the type of equipment used, and the sampling methods tried during the progress of the work. It must be emphasized that these programs were undertaken to secure subsurface data for engineering design and are in no way to be considered as research projects. Data obtained included classification of the soils in the upper "active" layer and in the permanently frozen "permafrost" classification of any segregated ice in the permafrost, water content, and in-place density of the several soils.

1955 Investigations

Scope of Work

In 1955, Fay, Spofford & Thorndike, Inc., was retained by the Eastern Ocean District, Corps of Engineers, to design several permanent-type buildings at the Frobisher Bay Air Base, a project requiring subsurface information needed to properly design building foundations. The area was known to be underlain with permafrost, but borings were required to determine the types of soil in the upper "active" layer which is seasonally frozen and thawed, the types of permanently frozen soil comprising the permafrost, and the existence of segregated ice in the permafrost.

The project, after being discussed in some detail with personnel of the Arctic Construction and Frost Effects Laboratory, New England Division, Corps of Engineers, resulted in a program consisting of 23 borings in an area of less than one-quarter of a square mile. Twelve of these borings were to be 30 feet deep and 11 of them 60 feet deep. Data required for foundation design were to be obtained by field testing, since some tests were better performed on frozen soil samples. However, it was planned to preserve typical samples in wax for laboratory testing in Boston in order to check field results and to obtain additional design data.

A contractor with previous experience in soil sampling in permafrost was engaged for the boring work. His method, employed during his previous Arctic work, entailed continuous drive sampling, with churn drilling substituted to advance the boring when drive sampling was stopped by harder material. In general, it was adopted for the 1955 program, with the added requirement that drive samples be taken every five feet when churn drilling.

Equipment

The drill rig used was a Bucyrus-Erie Model 20-W churn drill adapted for continuous drive sampling (see Fig. 1). The personnel included one superintendent, two drillers, and four drillers' helpers. This group was divided into two crews, each working 10 days a day. The superintendent organized the equipment and personnel, prepared boring logs, and generally supervised the work. Each crew was composed of one driller who operated the drill, and

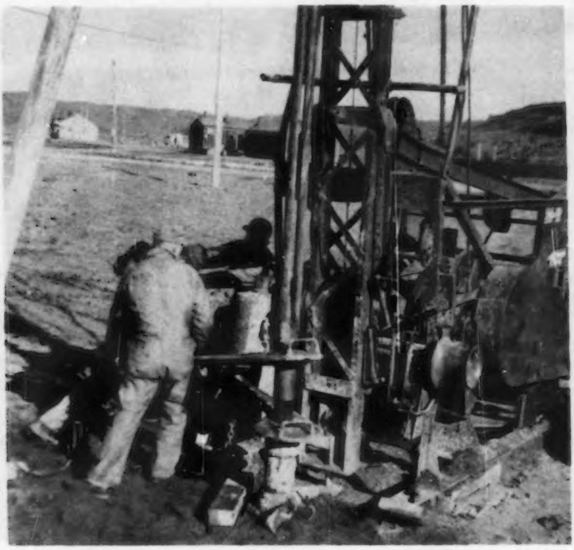

Fig. 1. Bucyrus - Erie Model 20W Churn Drill used in 1955 Program.

two helpers who made up the tool string, sharpened bits, and otherwise maintained the equipment. Due to the remote location of the site and relatively limited facilities for equipment repair, all of the helpers were qualified drillers and all of the men rotated jobs throughout the work.

The drilling tools were suspended by wire rope and, when drive sampling, included a rope socket, drill stem, drilling jars, core barrel, and drive shoe. The drilling jars consisted of two steel members interlocked to permit movement in the vertical direction only, this movement being about 30 inches maximum. A walking beam on the drill rig caused the upper part of the tool string to move vertically about 20 inches. A total weight of approximately 850 pounds was raised and dropped by this walking beam, the drilling jars being the part which transmitted the driving energy to the drive shoe. As the core barrel and drive shoe were driven into the ground, the wire rope was fed from a drum by a hand lever to permit continued penetration.

When spudding or churn drilling, the tools consisted of a rope socket, drilling jars with a 6-inch maximum travel, and a rock bit. The entire tool string was repeatedly dropped, breaking up stones and boulders in the way. Water was added to make a slurry of the pulverized stone and the hole bailed out.

The drill rig was skid-mounted and of such size and weight that moves between borings were made with a crawler-mounted tractor. The area in which the work was located was generally of low relief with a sandy topsoil in which a wheeled vehicle towing the drill rig might well have bogged down. Included as part of the equipment was a prefabricated tool shed. This also was mounted on skids for mobility and proved to be invaluable for keeping small tools readily available.

Fig. 2. Drive shoes after driving into gravelly soil. (1955 Program)

Sampling

All soil samples of the active layer and of the permafrost were obtained by drive sampling. However, at the start of the work, water was found above the permafrost table and caused caving of the sides of the hole by running in and thawing the permafrost. To protect the hole, a casing was driven into the permafrost sealing off the flow of ground water. As the boring was advanced, thawing progressed downward and the casing had to be redriven to maintain the seal.

Soil samples obtained by drive sampling varied from 3-1/2 to 4-1/2 inches in diameter. The drill was equipped with a piston type sample extruder to remove the samples from the core barrel. Those which were satisfactory for soil and ice classification and for simple field testing were obtained only in five-grained sands and silts. Occasionally, one was lost due to the hard back-driving required to free the sample spoon.

When coarse sands and gravels were encountered, little, if any, penetration was possible by driving. (Drive shoes were badly damaged in gravelly materials, as can be seen in Fig. 2). The operation was then changed to churning or spudding with drive sampling attempted every five feet. The only samples obtained of coarse-grained material were small chunks recovered by driving or bailing, but there were entirely unsuitable for classification or testing.

1957 Investigations

Scope of Work

In 1957, Fay, Spofford & Thorndike, Inc., was retained by the Corps of Engineers to design a new runway, taxiway and parking apron system at the

Frobisher Bay Air Base. The project involved widening of the existing runway from 150 feet to 200 feet, lengthening it from 6,000 feet to 10,000 feet, and adding a new taxiway and apron. Once again, subsurface information was required to design the proposed paving.

It was believed that the churn-drilling method of 1955 was not entirely satisfactory for boring work in the permafrost area, due particularly to the impossibility of proving rock or boulders. Canadian and United States literature on subsurface exploration in the Arctic was studied and a decision made to adopt core boring methods. Borings were to be taken within an area of several square miles. Again, data required for the paving design were to be obtained by field testing of recovered samples. But further laboratory testing was deemed unnecessary.

The contractor engaged for this work had no previous experience in subsurface explorations in permafrost areas. Core drilling procedures, proved successful in other Arctic areas and outlined in the literature, were discussed in detail with the contractor and with personnel of the Arctic Construction and Frost Effects Laboratory. The technique agreed upon involved several operations. As in the 1955 work, drive-sampling was to be used to advance the boring to the permafrost. A casing would then be driven into the permafrost to seal out the ground water. The boring would be completed by core-drilling, using a chilled salt solution as the circulating fluid to prevent thawing of the side of the hole and the sample. Continuous sampling was expected, including those of any boulders or rock which might be encountered.

Equipment

The equipment used in this work was a Sullivan-12 core drill, equipped to take cores from 2 inches to 4 inches in diameter and drive samples from 1-1/2 inches to 3 inches in diameter. (See Fig. 3.) The operating crew consisted of one driller and two helpers. A superintendent was on the job during the initial part of the work to expedite setting up and organizing the equipment.

When the equipment was set up for drive sampling, a 300-pound hammer falling 18 inches was set up to drive the sample spoon. Though larger spoons were used as much as possible, hard driving often required the use of the smaller ones.

During core-drilling operations, only the 2-1/8 inch NXM core barrel was used. At various times, different drill bits, core lifters and combinations of these were used in an effort to improve sample recovery.

The core drill rig was skid-mounted with a winch and cable and was light enough to permit maneuvering in rough terrain, under its own power, once a suitable anchorage was found or set. For long moves and on level terrain, the drill was moved about with a 2-1/2 ton truck, but in hilly or swampy areas, a winch and cable was substituted.

Sampling

Drive samples of the thawed, active layer and of the upper foot or so of the permafrost were obtained. These varied from 1-1/2 inches to 3 inches in diameter. Since no sample extruder was available, samples were knocked out of the sample spoon by rapping the side with a hammer or bouncing the end on a board.

Before coring operations were begun, the hole was sealed by a casing to keep out ground water and the circulating fluid was prepared. Initially, this

Fig. 3. Sullivan - 12 Core Drill used in 1957 Program.

was prepared from salt, water, and sea ice. By adding ice and constantly agitating the mixture, its temperature was reduced to about 30 degrees F. During coring, the salt solution was circulated and its temperature, after circulation, varied from 31 degrees F to 33 degrees F. No cores were recovered in several coring runs.

An attempt was made to reduce the temperature of the circulation fluid further by using ice prepared in refrigerators located at the base and kept at temperatures well below zero. Although a large amount of ice and a small amount of water were used, the resulting solution was only one or two degrees colder than the original, even after several hours of agitation. No cores were recovered using this colder solution.

Since drilling with liquids had resulted in unsuccessful core recovery, it was decided to try drilling with compressed air. To dissipate the accompanying heat, a long hose was laid on the ground between the air compressor and the drill rig. This gave some good cores in gravels, sands, silts and ice (see Figs. 4 and 5), but over-all results were erratic as may be seen in Table 1.

Due to the delays in obtaining equipment for drilling with compressed air and the subsequent breakdown of the compressor, most of the sampling actually was done by driving methods. The samples of fine-grained soils obtained were generally satisfactory for identification and testing. However, the hammering required to remove the samples from the spoon often broke them up and prevented some testing.

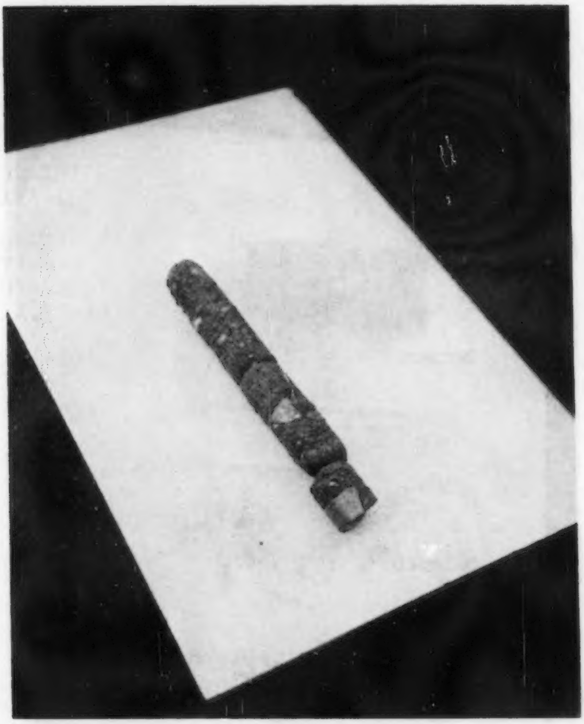

Fig. 4. Core of frozen gravelly sand obtained by core drilling with diamond bit and compressed air. (Hole # 24, 9'-12.5' depth, 1957.)

Comparison

General

The over-all rate of boring in 1955 with the churn drill was about 15 feet per 10-hour day. This included time spent in unpacking at the site, moving about the site, packing for return, and such factors as time lost due to bad weather and equipment breakdown. The cost of the borings, including mobilization, demobilization, and travel time to the site, was $42 per foot.

In 1957, the work actually done included 20 holes from 12 to 16 feet deep, 4 holes from 18 to 20 feet deep, and one hole 25 feet deep. The 60-foot hole originally planned went to a depth of 19 feet where rock was encountered and cored for an additional 6 feet. The rate of boring with the core drill was 8.1 feet per 10-hour day at a cost of about $44 per foot.

Equipment

From an economic standpoint, it appears that the cost of obtaining subsurface information is about the same, regardless of what method is used.

Fig. 5. Core of frozen silty sand with massive ice obtained by drive sampling. (Hole #18, 7'-8' depth, 1957.)

However, it is believed that core drilling is the more satisfactory method because of the mobility and versatility of the equipment used.

The core drill was much more easily handled for air transportation to the site. Once there, it was more easily moved about by truck or, in the event of rough or swampy terrain, by its own power.

Although the core drill was more mobile, its production rate was about half that of the churn drill. This was due to several reasons. The biggest loss of production was due to experimenting with drilling fluids in an effort to utilize the coring capability of the drill and the actual use of the core drill for drive sampling during the greater part of the job. Another factor which affected production was the time spent in fairly long moves between holes and the relatively short time spent in actually making the shallow borings.

Sampling

The churn drill obtained samples which were satisfactory for classification and testing only in fine-grained sands and silts. No suitable samples of coarse-grained materials were obtained and there was no way of proving whether coarse gravel or bedrock was encountered.

TABLE 1

CORE DRILL RECOVERY RESULTS

Hole	Depth	Soil	Bit	Recovery
18	9'-12'	-	Carballoy	0%
	12'-15'	SM	"	100%
24	6'-7'	SW	Diamond	70%
	7'-9'	SW	"	90%
	9'-12.5'	SP	"	85%
	12.5'-14'	-	"	0%
26	10'-12'	SP	Diamond	40%
	12'-13'	SW	"	100%
	13'-15.5'	SP	"	80%
29	12'-13.5'	SM	"	60%
	13.5'-16'	SM	"	75%
	16'-17.5'	SP	"	70%
	17.5'-20'	SP	"	85%
15	5'-7'	-	Diamond	0%
	7'-9'	SP	"	100%
	9'-11.5'	SP	"	50%
	11.5'-14.5'	SP	"	50%
	14.5'-16'	Boulder	"	75%
	16'-19'	SP	"	100%
	19'-25'	Granite	"	75%
17	7'-9.5'	SM	Carballoy	100%
	9.5'-12.5'	SM	"	65%
	12.5'-16.5'	SM	"	60%

NOTE: Drive sampling was used above the depths shown in the table. Drilling between the depths shown in the table was done using NXM double-tube core barrels and compressed air with bottom discharge bits.

Samples suitable for classification and testing were obtained by core drilling with air in all types of soil encountered, although the core recovery was unreliable. However, cores of boulders and bedrock were recovered, showing the capability of the core drill for proving the location of bedrock. When it was used for drive sampling in permafrost, only fine-grain soil samples were recovered.

The sample extruder mounted on the churn drill helped to produce samples that were of a good size for testing. With the core drill rig, the hammering used to remove the sample from both drive sample spoons and core barrels fractured the sample to such an extent that testing was limited on many samples.

Classification and Testing

The soil testing program was essentially the same for both years and consisted of determination of natural moisture content, in-place density, gradation, and Atterberg limits. Equipment for these tests, with replacements for fragile items, was shipped to the site and a laboratory was set up at the Base. One soils engineer coordinated the boring program and operated the soils laboratory.

Description and classification of the frozen samples were made immediately after recovery. The soil types were identified and described in accordance with the Unified Soil Classification System of the Corps of Engineers. The ice in the sample was described and classified in accordance with SIPRE Report 8 of the Snow, Ice and Permafrost Research Establishment, Corps of Engineers.

In addition to field testing, samples were preserved in wax for laboratory testing in Boston to check field results. The wax was tough enough to stand the rigors of transportation from the site to Boston and effectively prevented loss of moisture from the sample after the sample had thawed inside the wax coating. These samples also proved adequate for further laboratory tests to determine freezing and thawing characteristics.

Design

The permafrost has a tremendous bearing capacity for foundation design as long as it remains frozen. Any construction operation, whether for buildings or for paving, affects the thermal regime of the permafrost and may result in the raising (aggradation) or lowering (degradation) of the top of the permafrost. This change in the permafrost table is a function of such soil properties as moisture content and density.

In 1955, the Arctic Construction and Frost Effects Laboratory performed freezing and thawing tests on typical samples, as well as other tests to check field results. All of these data were correlated with the proposed building design and analyzed to predict the probable depth of thaw in the permafrost adjacent to the building.

Sieve analyses of the soils, as well as segregated ice found in the samples, indicated highly frost-susceptible material which would permit excessive settlement during summer thaw and excessive heaving during winter freezing. It was necessary, therefore, to prevent thawing of this material or to locate the building foundations well into the permafrost below any possible depth of thaw.

The foundation design recommended for this condition was spread footings on a gravel fill with an air space between the bottom of the building and the top of the gravel. The gravel fill is to serve as an insulating layer to present thawing of the frost-susceptible permafrost. The air space under the building will permit complete refreezing of the soil under the building each winter and prevent progressive degradation of the permafrost.

In 1957, again the problem was the possible lowering of the permafrost table under the paved areas. Most of the taxiway was to be on fill, which would act as an insulating layer and minimize change in the thermal regime of the permafrost. The runway and apron paving, however, would affect the permafrost directly below. The question arose whether the paving design should

provide complete or partial protection against permafrost degradation. Permafrost degradation would result in settlement of the pavement as the soil thawed. Partial protection would permit degradation of the permafrost and consequent damage to the pavement, which would require periodic maintenance.

If complete protection were to be provided, an eight-foot insulating layer of gravel would be required under the finished paving. If partial protection were considered adequate, the new paving could be overlaid directly on the existing paving, and periodic maintenance performed as required. In this particular location, it was recommended that partial protection be provided by an overlaid pavement.

SUMMARY

From the comparison of the drilling procedures described in this paper, it seems that core drilling is the better method for subsurface investigation in the Arctic if the full coring capability of the drill can be utilized. Experience to date indicates that the use of compressed air is the key to developing this capability. Factors which may affect core recovery are rate of air flow, cooling of air, size of core, speed of rotation, rate of feed, and type of bit. Also, there may be a depth limitation beyond which it is not possible to drill without excessive air requirements.

Once a satisfactory drilling method has been developed, it is believed that progress can be improved further by making the investigations in the early spring of the year. At this time, the ground is frozen to the surface, and there would be no need of drive sampling in the active layer or of casing the hole to seal out the ground water. The disadvantage is the effect of the weather on personnel and, consequently, on production.

In conclusion, it is believed that subsurface explorations in the Arctic are best accomplished by core-drilling methods. However, reliability of core recovery must be improved before core drilling can be considered fully satisfactory as a method of securing samples. The 1957 boring program indicates that further field experience, using compressed air with a core drill, will result in procedures that will ensure consistent recovery of permafrost cores.

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SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

CONSTRUCTION PORE PRESSURES IN AN EARTH DAM

C. Y. Li,[1] M. ASCE

SYNOPSIS

This paper presents an experience with construction pore pressures in a rolled earth dam. It is the study of a case history. As a result, a new method for predicting construction pore pressures is proposed.

INTRODUCTION

The existence and importance of construction pore pressures in an earth dam are generally recognized but a satisfactory method of predicting the pressures for design use has not yet been developed. An attempt is made here to analyze the data obtained from the construction of Quebradona Dam for the purpose of contributing information toward a better understanding of the pore pressures.

Quebradona Dam is a rolled earth fill structure about 100 feet high, located on a tributary of the Rio Grande about 40 miles from Medellin, Colombia, South America. It is part of the hydroelectric project for the City of Medellin. The construction started on June 1, 1956. Water was first impounded in the reservoir and the dam was put into full use on July 5, 1958, after completion of all the main features.

Under normal climatic conditions, the construction of Quebradona Dam would be particularly susceptible to the development of high pore pressures in the fill. The available silty material for embankment core had a natural water content of about 35 per cent and a natural density of about 82 pounds per cubic foot. The placement of embankment had to be carried out rapidly in order to fully utilize the relatively short annual dry seasons. Fortunately, the period of actual construction of the dam embankment coincided with the driest

Note: Discussion open until March 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2213 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 5, October, 1959.

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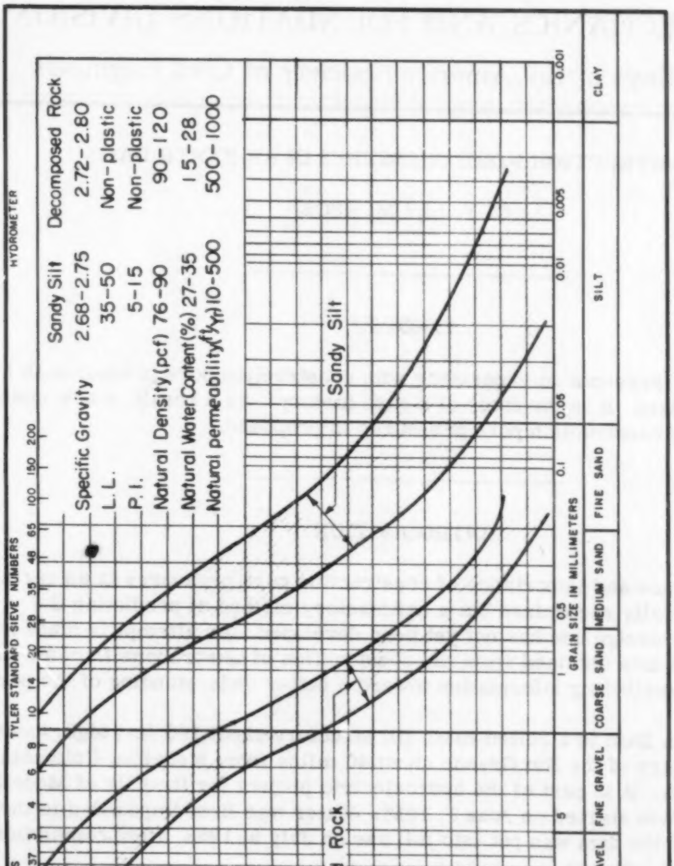


FIG. 1 GRADATION CURVES (NORMAL RANGES)

year on record. The soil placed was in a much drier condition than the normal natural water content. Consequently, the actual construction pore pressures are much lower than those estimated in the design. Nevertheless, the actual measured pore pressures and compressions in the embankment, and other information related to the setup of the pore pressures are interesting data worth studying.

Embankment Construction

Fill Materials

The embankment fill was constructed with two types of residual soil, silt and decomposed rock,[2] which are derived from the weathering of an igneous rock, quartz-diorite. The compacted silt, a relatively impervious soil (Darcy's k = 10 feet per year), was used as the core material. The compacted decomposed rock, being somewhat more pervious than the compacted silt (Darcy's k = 30 feet per year), was used as the shell material. The gradation curves and other general characteristics of the soils are shown in Fig. 1. The typical embankment section of the dam is shown in Fig. 2.

Compaction

The fill material was compacted by tamping rollers. The actual compacted density averages 96.2 pounds per cubic foot at 25.4 per cent water content for

2. The names do not adequately describe the soils, so they may be regarded for identification only.

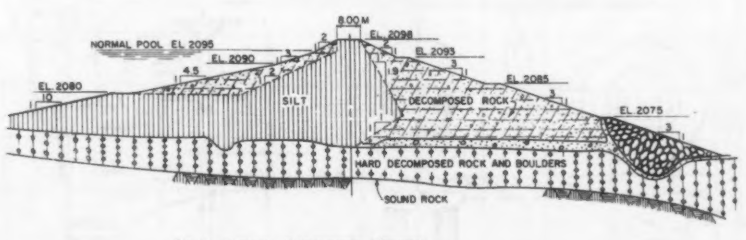


FIG. 2 – TYPICAL EMBANKMENT SECTION

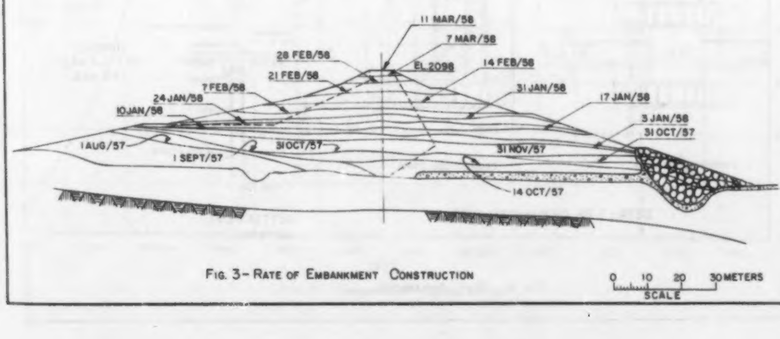


FIG. 3 – RATE OF EMBANKMENT CONSTRUCTION

silt and 105.7 pounds per cubic foot at 17.7 per cent water content for decomposed rock.

Rate of Construction

Most of the embankment was placed within a period of about 3 months (December 10, 1957 to March 15, 1958). During this period, the total height of the embankment placed was about 80 feet. The rate of embankment load increase was approximately constant (Fig. 3).

Test Apparatus in Dam Embankment

Three types of test apparatus were installed: (1) 15 piezometers were placed within and beneath the dam to measure pore pressures and hydraulic gradient; (2) one settlement installation, consisting of 9 crossarms 10 feet apart vertically, was installed at the point of intersection between the upstream edge of the dam crest and the original stream bed to record the embankment consolidation throughout the whole height of impervious silt core; and (3) 7 surface settlement points were placed on the downstream slopes of the dam to record the total post-construction surface settlement.

The installed test apparatus is similar to that used in the U. S. Bureau of Reclamation's earth dams.(1) The principal details of the apparatus are shown in Fig. 4. Plastic tubing was used to connect the piezometer tips with the measuring gages (Bourdon-tube type) located in the piezometer house on the downstream face of the dam. The equipment installed in the piezometer house includes a 1/4-inch standard brass pipe manifold, an electric water

Fig. 4 Test Apparatus

pump, an air trap, gages, needle valves and auxiliary fittings. The test equipment was purchased commercially in the United States.

Continuous readings from all the test apparatus have been recorded since their installation. They are found to be quite consistent and dependable. The readings were reduced into graphs and sketches which are presented in Figs. 5 to 13 inclusive. These figures are intended to be self-explanatory.

Observations

Embankment Consolidation

The record of embankment consolidation during construction is shown by Figs. 5, 6, and 7. Consolidation was rapid, practically coinciding with the placement of fill. Eighty-three per cent of the total primary consolidation had occurred by the time construction was completed. It is expected that virtually all of the remaining 17 per cent will have taken place within one year after construction. Fig. 6 shows that the field compression curves for the material at different crossarm intervals are not the same and vary according to the elevations of the material placed. Since the soil type and its placement conditions are practically unchanged throughout the full height of the settlement installation, this difference in compression defies a satisfactory explanation.

Construction Pore Pressures

The data concerning pore pressures are shown in Figs. 8 through 13 inclusive. The following observations regarding construction pore pressures have been made:

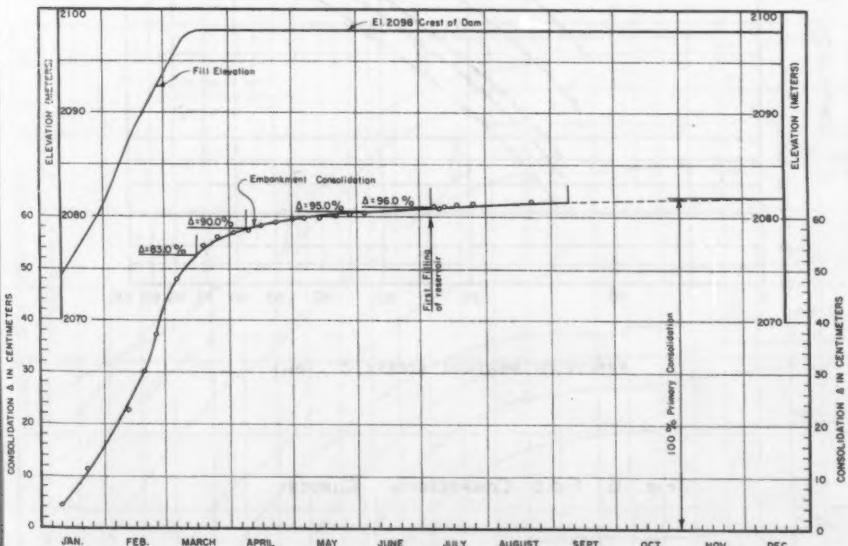


FIG. 5 EMBANKMENT CONSOLIDATION

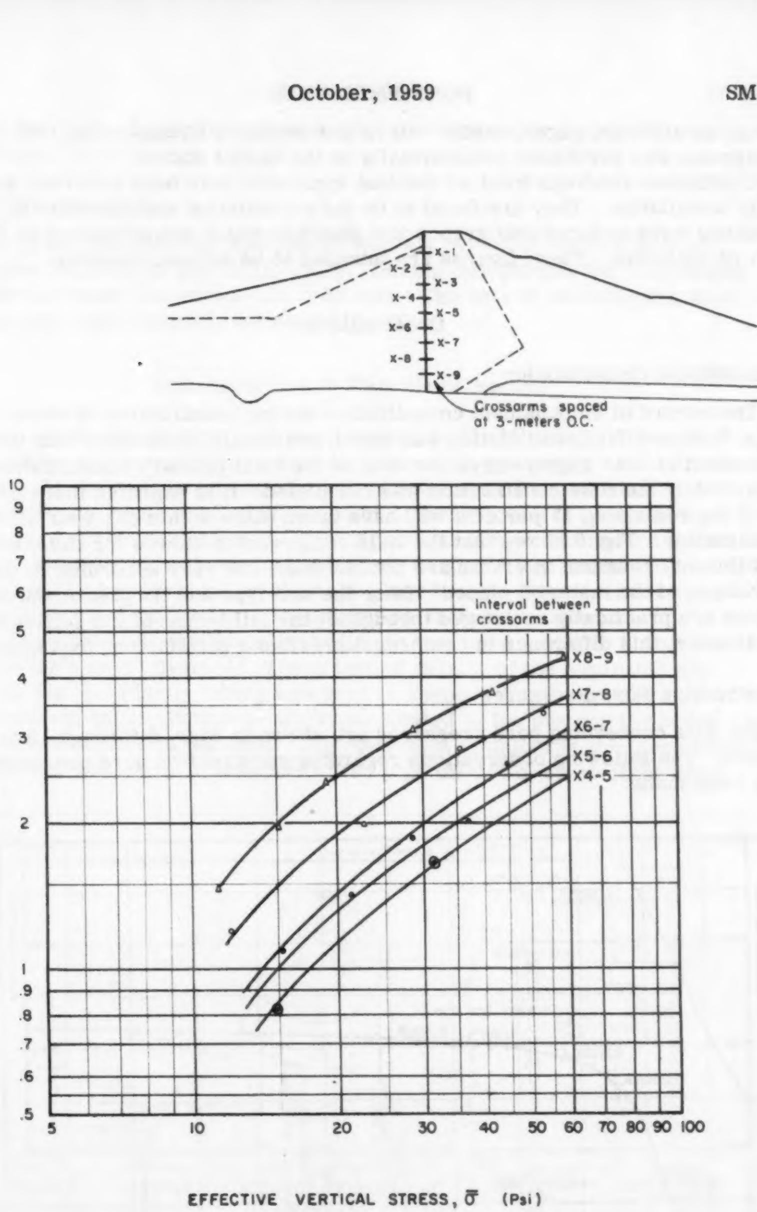


FIG. 6 FIELD COMPRESSION CURVES

1. **The rate of increase of pore pressures decreases with the total loading, as shown in Fig. 12. This may be attributed principally to the effect of drainage in the fill. For a fill allowing no drainage, the rate of increase of pore pressures increases with the loading until the condition when there is no free air in the soil voids; and then the rate of pressure increase equals to that of the loading.**
2. **Fig. 13 shows that the relationship between the measured pore pressure and the measured vertical strain is reasonably well defined. Thus, it clearly indicates that the pore pressures can be predicted from the laboratory compression test of the representative samples. The maximum construction pore pressure is less than 20 per cent of the total load under the existing conditions. (Fig. 12)**
3. **Both Figs. 12 and 13 show that the theoretical calculation of the construction pore pressure, assuming no drainage, is far from reality.**
4. **The dissipation of the construction pore pressure occurred rapidly. (Fig. 9)**
5. **The construction pore pressures developed in the decomposed rock fill are very small. (Figs. 8 and 9)**
6. **The drainage blanket is a very effective means of reducing construction pore pressures. (Figs. 8, 9, and 10)**

Discussion

Theoretical

The progressive addition of loading during the construction of a rolled earth fill is accompanied by the gradual process of volumetric decrease of

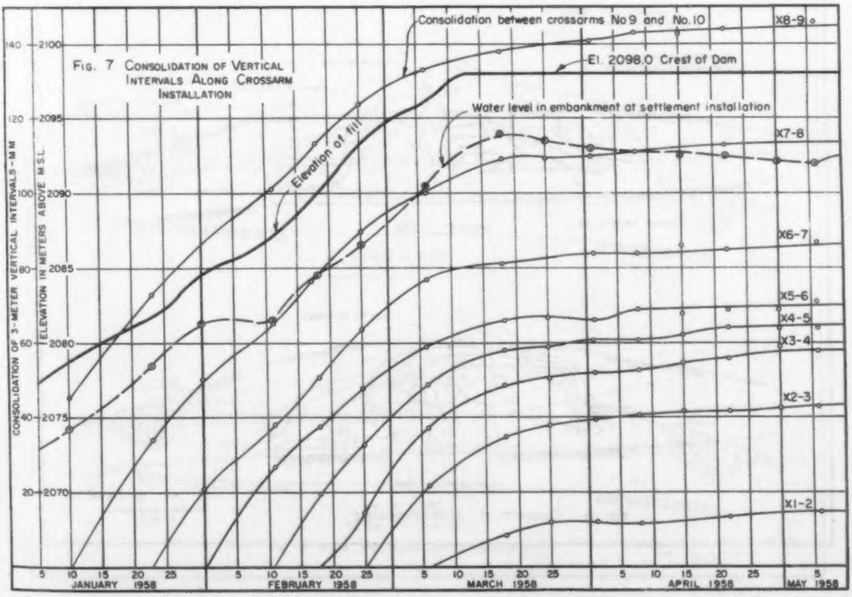


Fig. 7 CONSOLIDATION OF VERTICAL INTERVALS ALONG CROSSARM INSTALLATION

the underlying soil mass. Since a soil mass is made up of solid soil particles, water, and air, and assuming that the water and soil particles are incompressible, the volumetric decrease is the effect of the extrusion of air and water. If air and water can be expelled from the soil mass as fast as the volume change takes place, no pore pressure will develop. However, if a significant amount of time is required for the extrusion of air and water, the compression of the air, accompanied by the development of pore pressure in the soil mass, will result. As the excess air and water drains from the soil mass, the pore pressure dissipates. Therefore, the magnitude of the pore pressure developed in a rolled earth fill depends upon the following principal factors:

1. The physical characteristics of the soil mass:
   a. Compressibility
   b. Volumes of air and water
   c. Permeability
2. The time rate of construction.
3. The drainage provisions in the fill.

The determination of pore pressures by taking all the above factors into consideration is a very complex problem, the solution of which is possible only after making simplifying assumptions. The theoretical calculation of pore pressures, assuming no drainage or no dissipation, such as the derivation by Hilf(2,3) represents the solution of the problem under limited conditions.

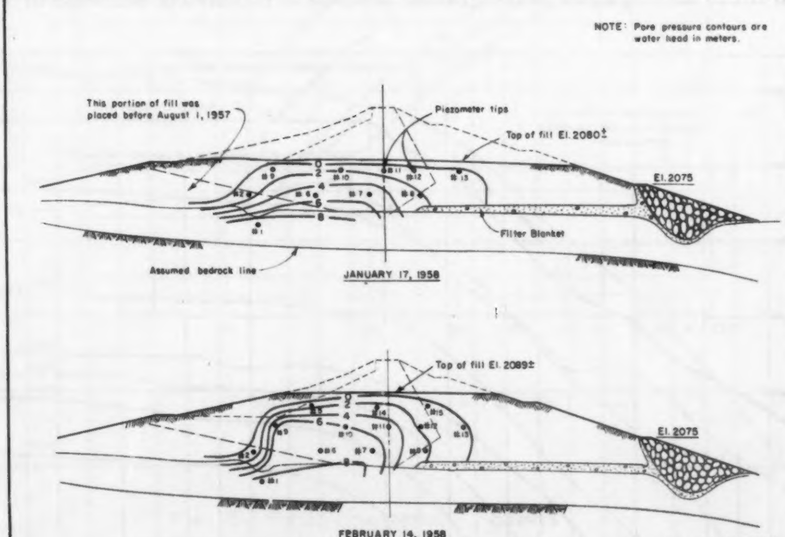


FIG. 8 CONSTRUCTION PORE PRESSURES

Experimental

The construction of Quebradona Dam may be viewed as a large scale experiment on pore pressures in an embankment fill under controlled conditions. There were many conditions experienced during construction that reduced the number of variables that influence the setup of pore pressures and, thus, simplified the analysis of a complex problem. The more important of these conditions were: The rate of increase in embankment loading was approximately constant; the soil placed in the zoned section consisted of the same type of material throughout the entire height of the dam; the compacted material had a limited variation both in moisture content and density; and the drainage characteristics of the core material at the location where the measuring instruments were installed was approximately uniform in all directions.

Estimating Construction Pore Pressures, Considering Dissipation

The pore pressure in a soil mass resulting from an increase in the total stress, without drainage, can be computed by the following equation:(2)

$$u = \frac{p_a \, \Delta}{(\bar{V}_a + H\bar{V}_w) - \Delta} \tag{1}$$

where u = Total pore pressure after compression minus atmospheric pressure, i.e., piezometer reading.

p_a = Air pressure, approximately atmospheric.

$\bar{V}_a$ = Volume of free air in the voids, in percentage of initial volume of soil mass.

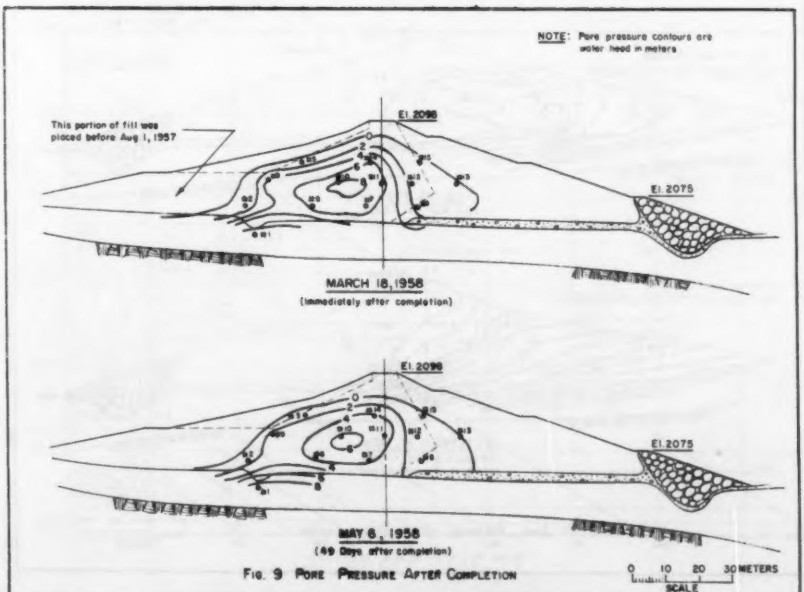


Fig. 9 Pore Pressure After Completion

$\bar{V}_w$ = Volume of water in the voids, in percentage of initial volume of soil mass.

H = Henry's constant of solubility of air in water by volume (0.0198 at 68° F).

Δ = Compression, or volume change, in percentage of initial volume of soil mass.

As reported by Hilf,[3] Eq. (1) gives only the theoretical air pressures in the soil voids. The pore pressure should be the algebraic sum of the air pressure and the capillary pressure due to surface tension of the pore water. The inclusion of the negative capillary pressure would reduce the computed pore pressure. The surface tension effect is neglected in this study because the observed piezometer pressures do not suggest its presence.

Actually, the drainage process of the soil mass sets in as soon as pore pressure develops. The drainage of pore fluids (water and air) dissipates the pore pressure at a rate depending upon the permeability of the soil mass, the time rate of construction and the drainage provisions in the fill. It is believed that this effect of drainage should generally be considered in estimating construction pore pressures even in very clayey soil, because any partial drainage would influence greatly the setup of pore pressures immediately afterwards.

An approximate method of determination of pore pressures to include the effect of pore pressure dissipation during the shut down period between construction seasons was developed by Bishop.[4] He computed pore pressures in the fill during the construction season by assuming no drainage, and then

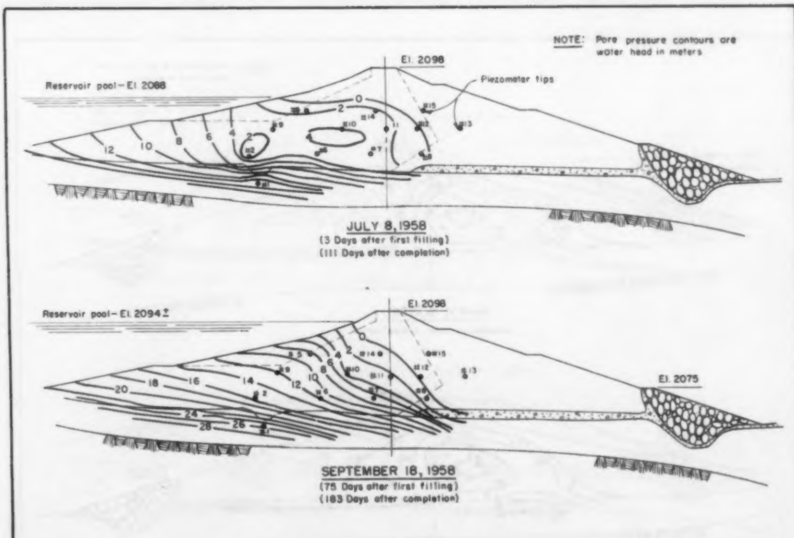


Fig. 10 Pore Pressure After First Filling

allowed a certain proportion of the pore pressures to dissipate at the end of the construction season before the application of embankment loading from the next construction season. The same process is repeated the number of times equivalent to the number of the loading stages or construction seasons.

Reasoning similar to that used by Bishop may be applied to obtain the total pore pressure in the earth dam to include the dissipation effect throughout the entire period of a single construction reason. The embankment loading is divided into an arbitrary number of increments. The pore pressure in the fill due to each load increment, without allowing drainage, is determined by Eq. (1). Then a certain chosen percentage of the computed pore pressure is allowed to dissipate at the end of each increment before the next begins. The same process is repeated the number of times equal to the number of the load increments.

Method of Computation

The method of computation is based on the following assumptions: (a) Eq. (1) is valid; (b) during dissipation, the mixture of air and water drained away has the same proportion by weight as that remaining in it, i.e., the degree of saturation remains unchanged before and after dissipation; and (c), the actual dissipation is a continuous, gradual process. The proposed method of calculation is an approximation of that process by steps.

The amount or percentage of pore pressure dissipation after each increment of effective stress must be assumed. This procedure requires good judgment which depends on an understanding of the soil characteristics, the effects of the construction rate and the drainage features. It may be considered as an empirical constant the accuracy of which depends entirely on judgment.

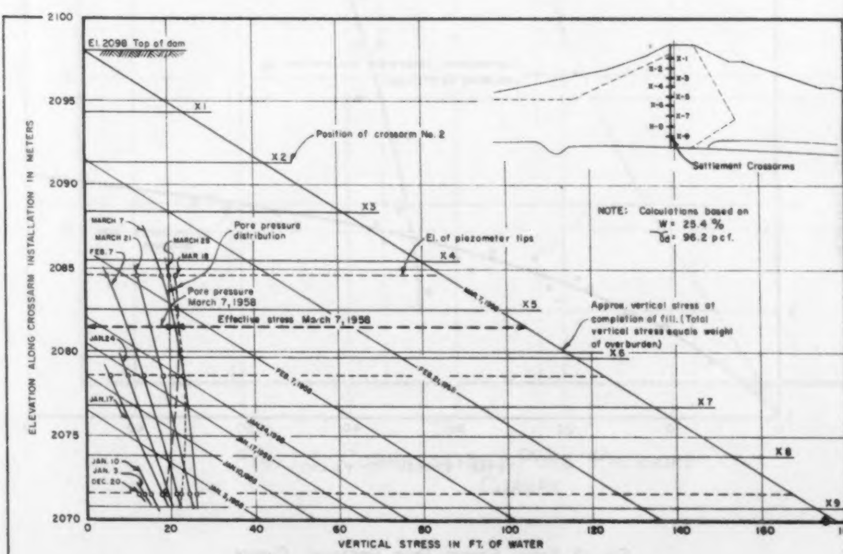


FIG. 11 DISTRIBUTION OF VERTICAL STRESSES

In Fig. 14, the curve showing the relationship between the effective stress, $\bar{\sigma}$, and volume change, Δ, may be obtained from laboratory consolidation tests. For an arbitrary assumed increment of $\bar{\sigma}_1$, the Δ_1, is obtained. The pore pressure u_1, corresponding to Δ_1, with no drainage, can be computed from Eq. (1) and point 1a is obtained. Next, the pre-assumed percentage of dissipation after the increment in effective stress is applied. Since the effective stress is increased by the same amount of pore pressure dissipation, the corresponding additional compression results. Point 1b is obtained. Thus, increment 1 ends and increment 2 begins.

From the $\bar{\sigma}$ vs. Δ curve, the Δ_2 corresponding to $\bar{\sigma}_2$ is obtained. To compute pore pressure from Δ_2, without allowing drainage, the initial conditions of Eq. (1) must be modified. The initial air pressure, p_a, will be the atmospheric pressure plus the undissipated pore pressure. The total air volume, $\bar{V}_a + H\bar{V}_w$, can be computed by the equation:

$$\bar{V}_{a_1} + H\bar{V}_{w_1} = (\bar{V}_{a_0} + H\bar{V}_{w_0})\left(\frac{p_a}{p_a + p_{1b}}\right)\left[\frac{100n - \Delta_{1b}}{100n - \dfrac{(\bar{V}_{a_0} + H\bar{V}_{w_0})u_{1b}}{p_a + u_{1b}}}\right] \qquad (2)$$

The derivation of this equation is shown as Fig. 15. Using Eq. (1) again and the new initial conditions, the pore pressure u_2 (point 2a), corresponding to the second increment of effective stress, can be computed. Then let the pressure dissipate to a desired value. Additional compression follows the dissipation and may be obtained from the $\bar{\sigma}$ vs. Δ curve. Point 2b is thus determined. This is the end of increment 2 and the beginning of increment 3.

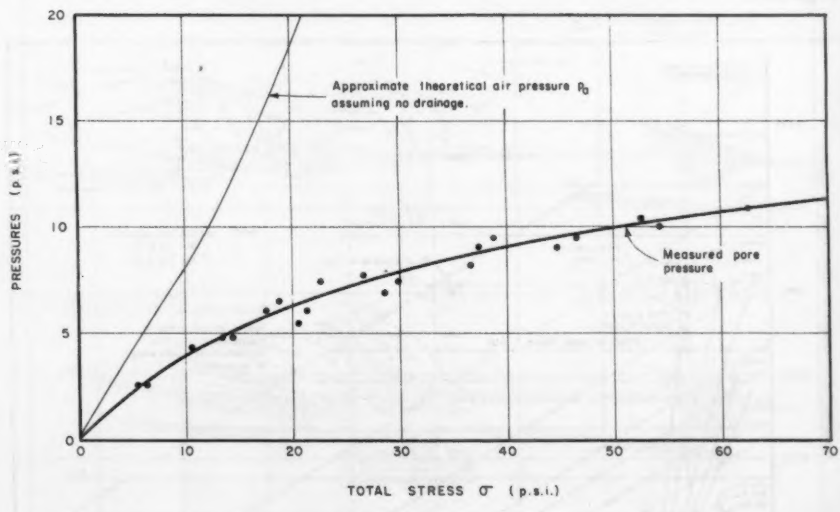


FIG. 12 TOTAL STRESS–PORE PRESSURE CURVES

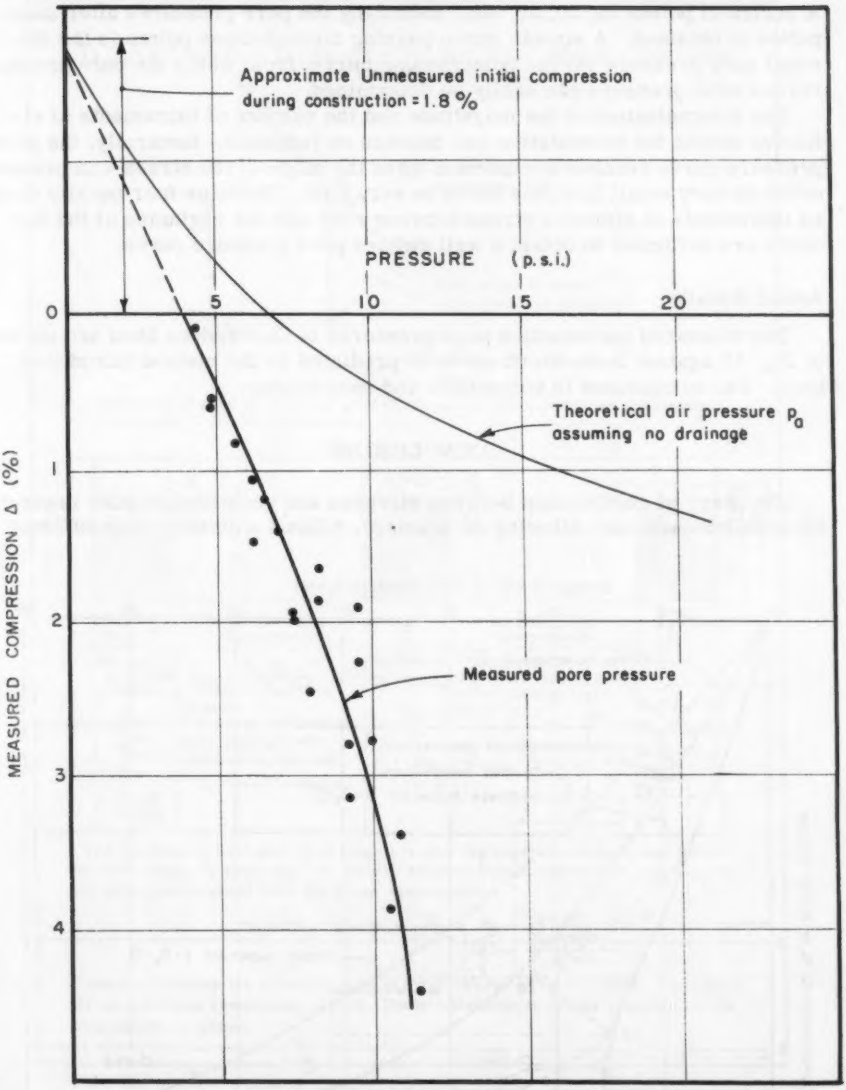


FIG. 13 COMPRESSION - PORE PRESSURE CURVES

The same process is repeated for increment 3, for increment 4 and so on. A series of points 1b, 2b, 3b, etc., indicating the pore pressures after dissipation is obtained. A smooth curve passing through these points is the desired pore pressure versus compression curve, from which the pore pressure versus total pressure can easily be determined.

The determination of the magnitude and the number of increments of effective stress for computation use depends on judgment. Generally, the pore pressure curve reaches a maximum when the slope of the stress-compression curve is very small (i.e., the curve is very flat). Three or four equally divided increments of effective stress between zero and the beginning of the flat curve are sufficient to obtain a well defined pore pressure curve.

Actual Results

The measured construction pore pressures at Quebradona Dam are plotted in Fig. 16 against those which could be predicted by the method introduced here. The comparison is compatible and encouraging.

CONCLUSIONS

The curve of relationship between stresses and construction pore pressures for a rolled earth fill, allowing no drainage, follows a distinctively different

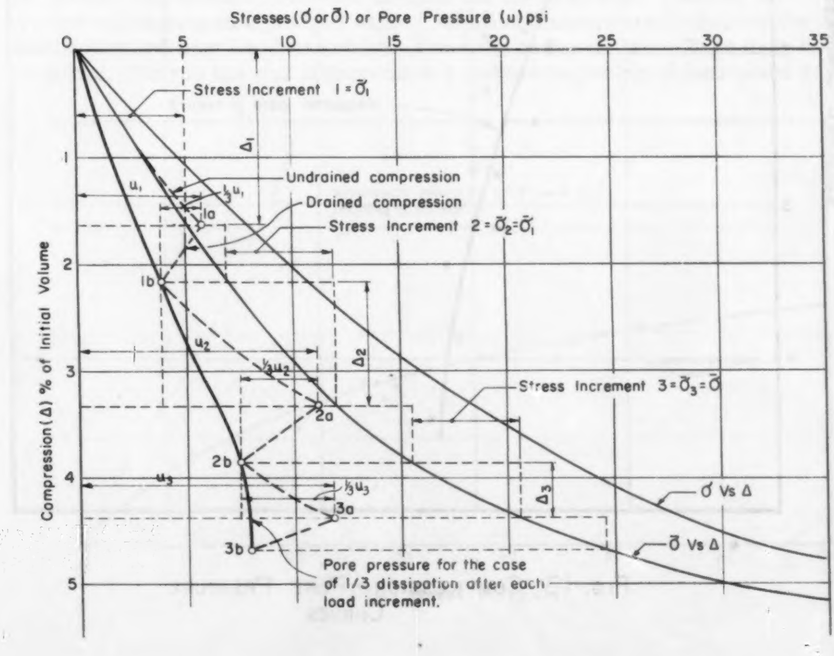
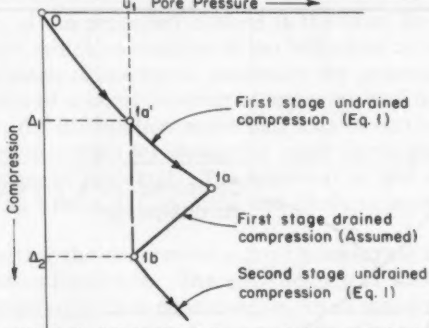


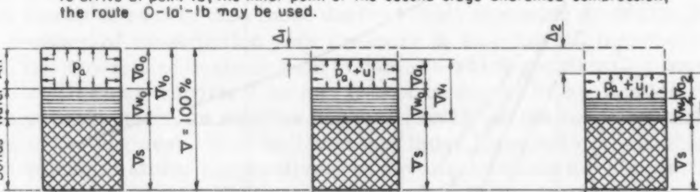
FIG 14 DERIVATION OF PORE PRESSURES ALLOWING DRAINAGE

Fig. 15

To Derive Constants in Eq. 1 For Use in The Next Stage Undrained Compression



To arrive at point 1b, the intial point of the second stage undrained compression, the route O-1a'-1b may be used.



Initial Point O	Undrained Compression	1a'	Drained Compression	Final Point 1b
All initial conditions known ∇a_o ∇w_o ∇v_o ∇s_o p_a $n_o = \frac{\nabla v_o}{100}$	$u_1 = \frac{p_{a_o}\,\Delta_1}{(Va_o + H\nabla w_o) - \Delta_1}$ in which u_1, being the assumed pore pressure after dissipation, is known. Δ_1 may be computed.	$\nabla a_1 = \nabla a_o - \Delta_1$ $\nabla v_1 = \nabla v_o - \Delta_1$ $S_1 = \frac{\nabla w_o}{\nabla v_1}$ $= \frac{\nabla w_o}{\nabla v_o - \Delta_1}$	The proportion of air and water leaving sample is the same as that remaining, or $S_2 = S_1$ Δ_2 can be obtained from $\Delta - \bar{\sigma}$ Curve.	$p_{a_2} = p_{a_o} + u_1$ $\nabla v_2 = \nabla v_o - \Delta_2$ $\nabla w_2 = S_1 \nabla v_2$ $\nabla a_2 = \nabla v_2 - \nabla w_2$

The condition at final point 1b of stage 1 is also the condition of the initial point of the next stage. To apply Eq. 1 for second stage undrained compression, $p_{a_2} = p_{a_o} + u_1$ and after rearrangment from the above relationships,

$$\nabla a_2 + H\nabla w_2 = (Va_o + H\nabla w_o)\left(\frac{p_a}{p_a + u_1}\right)\left[\frac{100n_o - \Delta_2}{100n_o - \frac{(\nabla a_o + H\nabla w_o)\,u_1}{p_a + u_1}}\right]$$

in which all volumes are expressed in % of initial volume of the soil mass. The origin for the second stage compression is at 1b. Similar relationships can be repeated for the third, fourth..... stages.

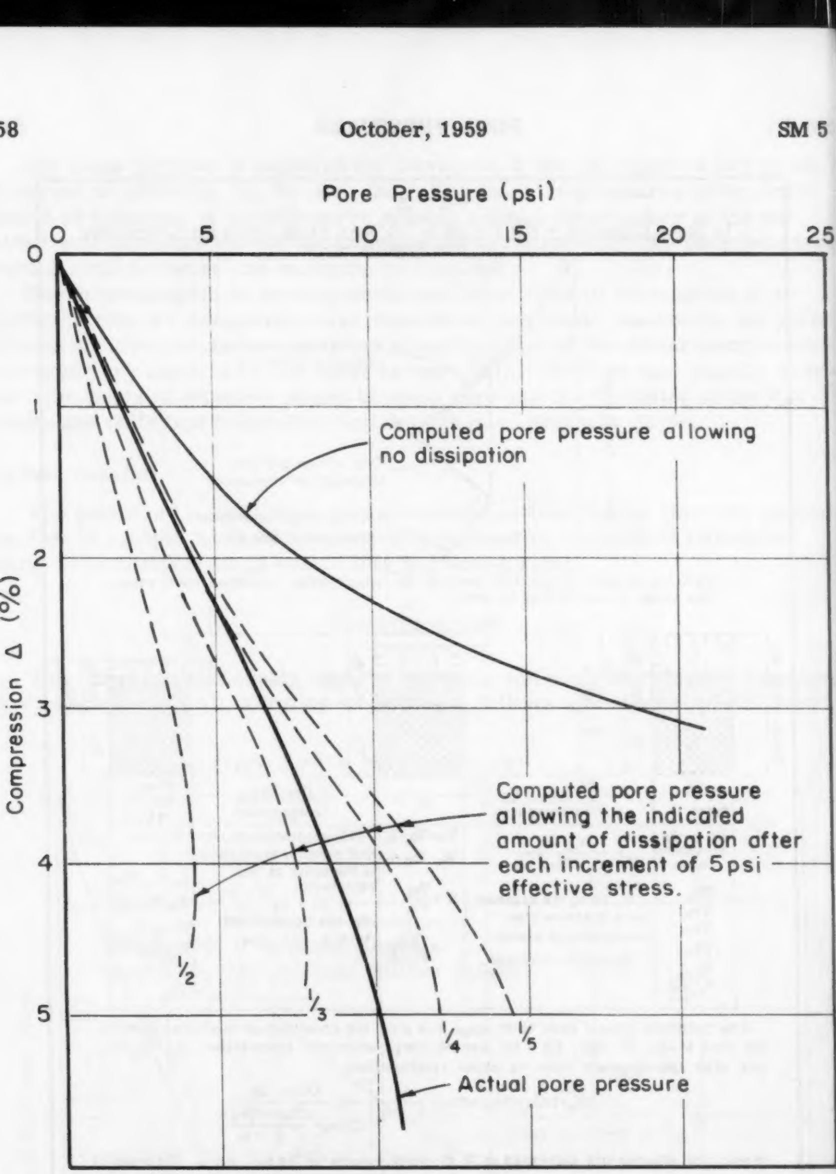


FIG. 16 COMPARISON OF ACTUAL AND COMPUTED PORE PRESSURES

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pattern from that allowing drainage, even a very small amount of drainage. The effect of drainage on the construction pore pressure is cumulative and is too great to be ignored under high embankment loading stresses, even in a very clayey fill.

The obvious defect of the proposed method is the need for assuming the percentage of drainage, which is similar to the selection of an empirical constant. The compilation of empirical constants for general use can only come from the analysis of a large number of experimental data under variable conditions. However, the dissipation value that may be arrived at from this study can be of immediate value for designing other earth dams to be constructed of similar types of material. The material in this study is a very common soil occurring widely in Colombia and likely in many other tropical areas.

A method of predicting the construction pore pressures, including the effect of drainage, is described here. The significance of such a method is that for most soils encountered in dam construction, even though the amount of drainage may be inaccurately assumed, the calculated pore pressures would be much closer to reality than those derived from assuming no drainage at all.

The problem of construction pore pressure in an earth fill is complex. At present, the problem of designing earth dams in which construction pore pressures will be an important factor, cannot be solved by application of precise design techniques; its solution depends greatly on the judgment and experience of the engineer. It is believed that those faced with this problem should continue to strive cooperatively to develop methods for simpler and more realistic solutions. This paper is a step in that direction.

ACKNOWLEDGMENTS

Dr. Oscar Baquero P., General Manager of Empresas Publicas de Medellin, the owner of the Quebradona Dam, cooperated by authorizing the installation of the test apparatus in the dam. Gannett Fleming Corddry and Carpenter, Inc., Harrisburg, Pennsylvania, were the design engineers and also the construction supervising engineers in a joint venture with Integral, Ltd. of Medellin, Colombia. The author was the engineer in charge of design and served as the resident engineer during construction. He is particularly indebted to Dr. Oscar Mejia V. of Integral, Ltd., for his generous cooperation and for the valuable suggestions he made during the course of the study.

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RATE OF CONSTRUCTING EMBANKMENTS ON SOFT FOUNDATION SOILS

Herbert L. Lobdell,[1] M. ASCE

SYNOPSIS

Procedures for predicting and controlling the rate of construction of embankments on soft weak foundation soils are presented. Shear strength, the key factor in such problems, is related to consolidation and effective stresses in the foundation soil. An illustrative example of a hypothetical embankment stability problem follows, and the assumptions used in such an analysis are discussed.

INTRODUCTION

One of the most difficult problems confronting soils engineers who are engaged in the planning and design of earthworks and embankments is that of predicting the safe rate at which embankments, which must be constructed over deposits of soft cohesive soils, can be built up without overstressing the foundation soils and causing large-scale shear failures. The problem occurs frequently in highway design, where fills, frequently ranging from 25 to 50 feet in height, must be built across soft marsh or swamp deposits. The natural shear strength of such deposits is usually too low to provide adequate stability against shear failure if the fill is placed according to a normal construction schedule. An estimate of the rate at which fill can be placed, even if only approximate, is therefore a very important consideration in scheduling construction operations and in meeting time limits for construction projects.

The stability of embankments founded on soft foundation soils depends on several factors which include: (1) the height, weight, and slopes of the embankment; and (2) the depth and strength of the foundation soil. The strength

Note: Discussion open until March 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2214 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 5, October, 1959.

1. Soils Engr., Greer Eng. Associates, A Div. of Woodward-Clyde-Sherard and Associates, Montclair, N. J.

of the foundation soil, in turn, is a function of the ever-changing degree of consolidation resulting from the imposed embankment load. Another variable is introduced in the changing height of the embankment as it is constructed. The problem of predicting the stability of an embankment at any time during or following construction therefore involves many variables and is one which can be solved only by a step-by-step and trial-and-error process.

To date, no rational approach to this problem of predicting a safe rate of filling of embankments on soft foundation soils has been offered. Although piezometers have been installed on many projects of this type as a means of controlling the safe rate of filling during construction, it is believed that judgment, experience, and rule-of-thumb methods have frequently been employed to govern the rate of filling in lieu of rigorous stability analyses based on pore pressures and effective stresses along potential shear planes. It is the intent of this paper to present a systematic procedure which can be used when dealing with problems of this nature.

Field Investigation

The first step in an investigation for a project of this kind is to lay out a boring program to determine the depth and lateral extent of the soft foundation soil deposit to be crossed. A sufficient number of representative undisturbed samples should be obtained at various depths to enable the pertinent physical properties to be determined in the laboratory. Disturbed samples from additional borings will permit correlation of soil properties over the site. Supplemental methods of exploration, such as jet probings or wash-borings can often be utilized economically to fully describe the contours of the bottom of the deposit.

Laboratory Testing Program

The physical characteristics of the foundation soil, particularly those of strength and consolidation, should be thoroughly investigated in the testing phase of an investigation. From the time curves of the consolidation tests, the coefficient of consolidation c_v should be computed. The pre-consolidation load, which affects the natural (or in situ) shear strength, should be determined from the pressure vs. void-ratio curves.

The shear strength of the foundation soil should be considered in two phases: first, its natural strength; and secondly, its potential strength. The natural strength at various depths can be determined from unconfined compression tests; from unconsolidated-undrained, or consolidated-undrained triaxial compression tests in which the samples are subject to stresses that existed in the natural state; or possibly by field vane tests. The potential strength is a function of the angle of internal friction for effective stresses (effective angle of friction) which can be obtained from a series of consolidated-undrained (Q_c) triaxial compression tests with pore pressure measurements. It has been found that essentially the same angle of friction[1,2] or one having a slightly lower value[3] is obtained from consolidated-drained tests, but these tests are generally impractical because of the time required to perform them.

The apparent angle of internal friction, which is obtained from consolidated-undrained triaxial tests in which only the total stresses are known, is applicable in stability analyses when the foundation soil is fully consolidated and is

not referred to further in this paper. There has been a great deal of misunderstanding and misconception in the past regarding the use of apparent and effective angles of internal friction in estimating shear strength. Although it is not the intent of this paper to discuss the application of these respective angles of friction in detail, it is hoped that the following will emphasize the point that the effective angle of internal friction should be used when computing the shear strength of partially consolidated soils, or soils in which excess pore pressures exist.

Relationship Between Effective Overburden Stress and Shear Strength

From the basic shear strength equation, which assumes a linear relationship between shear strength and normal stress,

$$\tau = \bar{\sigma} \tan \phi_e$$

where τ = shear strength

$\bar{\sigma}$ = effective normal stress

ϕ_e = effective angle of internal friction and the triaxial stress relationships shown in Fig. 1, the shear strength can also be expressed as

$$\tau = \frac{\bar{\sigma}_1}{2} \left[1 - \tan^2 (45 - \frac{\phi_e}{2})\right] \cos \phi_e \qquad (1)$$

By substituting a value for ϕ_e the equation simplifies to terms of τ and $\bar{\sigma}_1$. For example, if $\phi_e = 30°$, the equation becomes

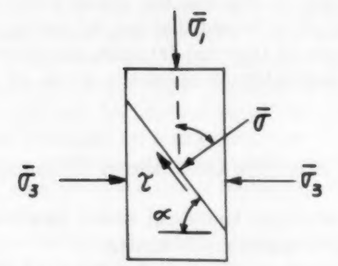
$$\tau = 0.29\ \bar{\sigma}_1$$

Plots of the ratio τ to σ_1 from triaxial test data in the range of shear failure have been found to check such an equation closely.[2,4]

The above equations are based upon frictional resistance, without regard to cohesion. Whether the tangents to a series of Mohr's effective stress circles for undisturbed cohesive soils should be drawn through the origin (assuming that the strength above the tangent up to the range of preconsolidation is cohesion), or whether it should be drawn above the origin according to the classic Coulomb conception, appears to be a matter open to question. A series of stress circles often deviate somewhat from an average or mean tangent, due possibly to variations in sample properties or disturbance; in such cases, it is difficult to determine which of the above interpretations to use. For normally consolidated soils, there is little difference between drawing the tangent to a series of Mohr's stress circles through or just above the origin.

The key to analyzing the stability of an embankment, for the purposes of this paper, lies in expressing the shear strength of the foundation soil in terms of the effective overburden or vertical stress. In nature, the effective overburden stress on an element of soil below the ground surface can be represented by $\bar{\sigma}_1$, the effective major principle stress. Based on this premise, the effective vertical or overburden stress can be computed to any depth if the weight of overburden and the pore pressure are known. Subsequently, the shear strength of an element of soil beneath the surface can be expressed in terms of the effective vertical or overburden stress.

(a)

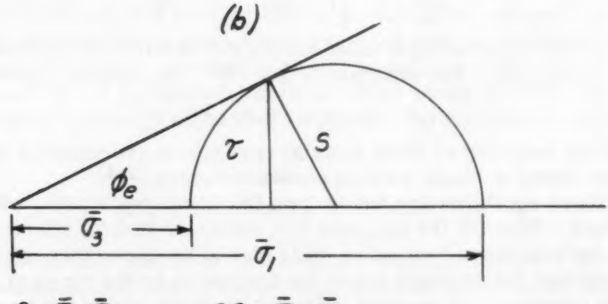


α = Max. angle of obliquity = angle of rupture

$= 45 + \frac{\phi_e}{2}$

ϕ_e = Effective angle of internal friction

(b)



$$S = \frac{\bar{\sigma} - \bar{\sigma}_3}{2} \qquad 2S = \bar{\sigma}_1 - \bar{\sigma}_3$$

$$\bar{\sigma}_3 = \bar{\sigma}_1 \tan^2\left(45 - \frac{\phi_e}{2}\right) \text{ from (a)}$$

$$2S = \bar{\sigma}_1 - \bar{\sigma}_1 \tan^2\left(45 - \frac{\phi_e}{2}\right) = \bar{\sigma}_1 \left[1 - \tan^2\left(45 - \frac{\phi_e}{2}\right)\right]$$

$$S = \frac{\bar{\sigma}_1}{2}\left[1 - \tan^2\left(45 - \frac{\phi_e}{2}\right)\right]$$

$$\tau = S \cos \phi_e = \frac{\bar{\sigma}_1}{2}\left[1 - \tan^2\left(45 - \frac{\phi_e}{2}\right)\right] \cos \phi_e$$

FIGURE 1 EFFECTIVE STRESSES IN TRIAXIAL COMPRESSION AT FAILURE

Design Procedure

The procedure of establishing a rate of filling essentially is one of considering the embankment to be built up in increments, as shown in Fig. 2; and finding the cumulative effect of the increments on the foundation soil, particularly along potential failure arcs, during the course of construction. Stability analyses are performed at various stages of filling to determine when the foundation soil, under the cumulative lifts of fill, has strengthened sufficiently to permit further increments of fill to be added.

After the consolidation and strength properties of the foundation soil are known, the first step in the actual analysis of the problem is to draw up an average time-consolidation curve for the foundation soil. Fig. 3 is the time-consolidation curve for the example presented later in the article which follows from the theory of consolidation where

$$t = \frac{T H^2}{c_v}$$

t = time

T = time factor

H = length of drainage path

c_v = coefficient of consolidation ·

Fig. 4 is the graphical presentation of the theoretical process of consolidation which aids in finding the per cent consolidation at various depths in the foundation soil.

The next step is to find the height of fill that the foundation soil will safely support, assuming the fill to be placed as rapidly as possible. This stability analysis can be made by a slip-circle analysis, or more simply by using the elastic method of analysis from Jürgenson's tables for terrace loading,(5) if the top of the embankment is relatively wide.

The remainder of the proposed fill should be divided into lifts or increments (preferably equal) and considered as strip loads as shown in Fig. 2. Sketches of each increment of fill and its induced vertical stress distribution

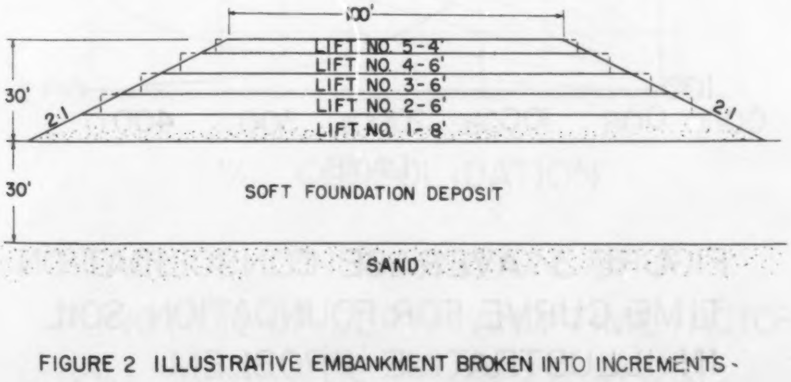


FIGURE 2 ILLUSTRATIVE EMBANKMENT BROKEN INTO INCREMENTS OR LIFTS

patterns should be drawn in relation to the embankment as a whole and so that all potential failure arcs are covered by the stress patterns as shown in Fig. 5. A sufficient number of copies of each increment and its induced stress pattern should be made to take care of the number of lifts of fill used and the number of stability analyses to be made.

The per cent of consolidation and subsequent increase in effective vertical stress for the foundation soil layer should be computed at different depths under each of the increments of fill. If only one height of increment is used throughout, then only one set of consolidation computations is necessary.

It has been stated that the shear strength τ can be expressed as a function of $\bar{\sigma}_1$ the effective vertical stress. Therefore an increase in effective vertical pressure in the foundation soil, due to embankment loading, will cause an

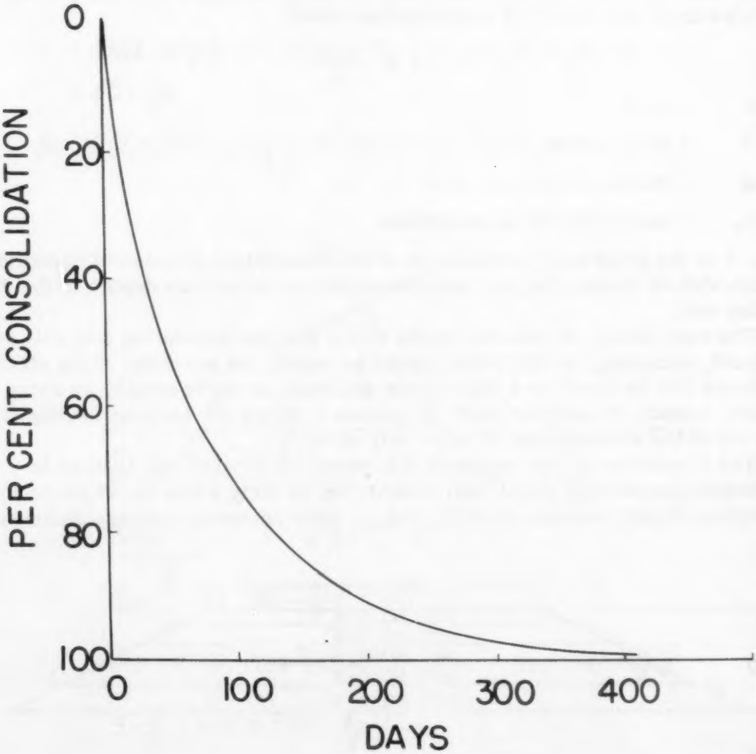


FIGURE 3 AVERAGE CONSOLIDATION-TIME CURVE FOR FOUNDATION SOIL IN ILLUSTRATIVE PROBLEM. (DOUBLE DRAINAGE)

increase in shear strength. The vertical stress at any point beneath the embankment depends upon the height and slope of the embankment and the depth of the point beneath the embankment. The imposed vertical stress of the full embankment must therefore be corrected by a stress coefficient according to position beneath the embankment (see Fig. 5). If we call increase in shear strength $\Delta\tau$, then at any point

$$\Delta\tau = (K) \quad (\Delta\bar{\sigma}_1) \quad (I_p)$$

where K $\quad$ = coefficient dependent upon value of ϕ_e (see Eq. (1))

$\Delta\bar{\sigma}_1$ = increase in effective vertical stress

I_p = vertical stress coefficient

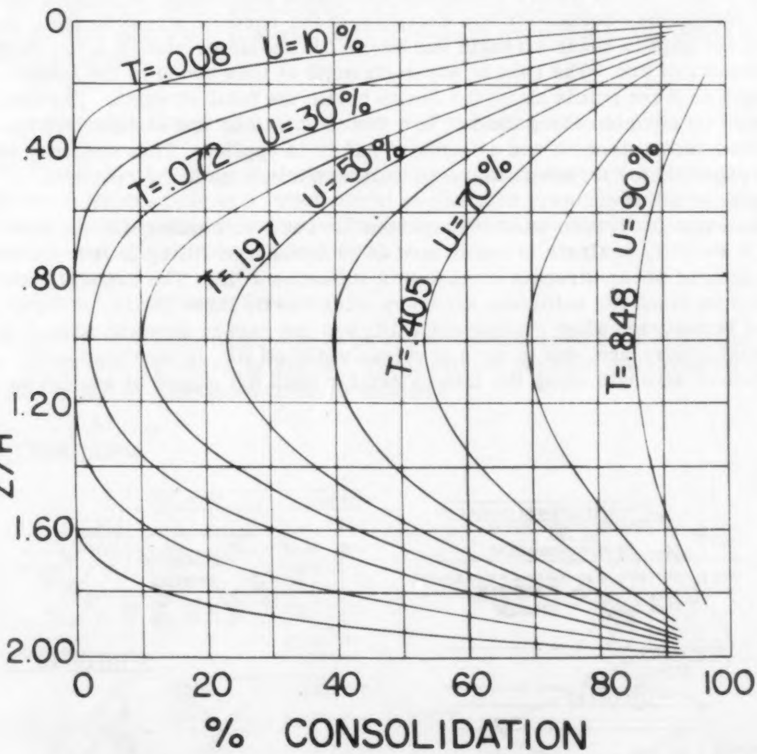


FIGURE 4 CONSOLIDATION AS A FUNCTION OF DEPTH AND TIME FACTOR (FROM THEORY OF CONSOLIDATION)

If we call the initial or natural shear strength at any point in the foundation soil C, then the total shear strength could be expressed as the sum of the initial or natural strength and the sum of strength increases due to embankment loading, or

$$\tau = c + \sum \Delta \tau$$

$$\tau = c + \sum \; (K \Delta \bar{\sigma}_1)\;(I_p)$$

The next step is to find the time required for sufficient consolidation and strengthening under the first lift to permit placing of the second lift with safety. This time should include the average placing time of the first lift plus a waiting period. A stability analysis must be performed to determine the most critical failure arc and to compute the average required shear strength. The potential arc of failure within the foundation soil should be divided into equal increments because of the variation in the vertical stress beneath the slope; the gain in shear strength due to the lift should be plotted along these increments of arc. The gain in shear strength is then added to the initial strength at these points along the arc to obtain the total strength. The total strength increments along the arc are then summed up and compared with the required shear strength and a factor of safety is applied. This step may have to be repeated for different assumed waiting periods until the required strength is achieved.

The same procedure must be repeated for each succeeding embankment lift. A stability analysis is performed at each stage of filling to find the average required shear strength to sustain a subsequent lift. The critical failure arc can be found for sufficient accuracy after two or three trials. A waiting period is assumed after placing each lift; and the gain in strength along the potential failure arc, due to each previously placed lift, is then computed. The gain in strength along the failure arc for each lift should be plotted on a

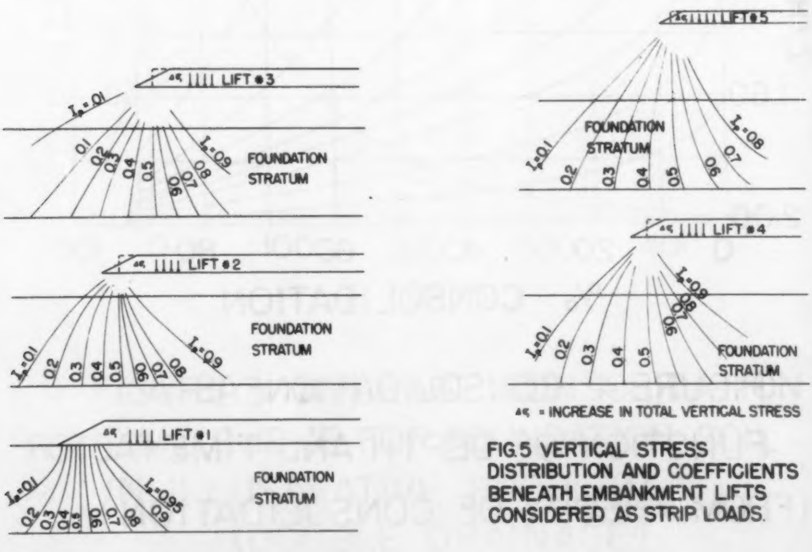


FIG. 5 VERTICAL STRESS DISTRIBUTION AND COEFFICIENTS BENEATH EMBANKMENT LIFTS CONSIDERED AS STRIP LOADS.

separately prepared sketch similar to those shown in Fig. 6, and should include the following:

(1) the potential failure arc;
(2) vertical stress pattern; and
(3) increase in strength with depth in the foundation soil assuming the vertical pressure is equal to the full height of the fill.

Sketches of each lift for each stage of filling can be drawn quickly on tracing paper.

The gains in shear strength along the increments of the arc for each lift are summed up and added to the initial strength to obtain the total strength of the foundation soil.

A plot of height of fill against time, as shown in Fig. 7, should be made during the analysis to serve as a record of the time of placing of each lift and the length of waiting period; it should also provide an approximate schedule for filling operations.

Example Illustrating Procedure of Predicting Rate of Filling

To illustrate the foregoing procedure of predicting the rate of filling of embankments, an example of a hypothetical problem is presented. The example represents a typical problem which could be encountered in the design and constructions of a highway embankment.

Conditions of Illustrative Problem

A 30-foot high embankment is to be constructed across a 30-foot depth of soft clayey silt, as shown in Fig. 8. The pertinent physical characteristics of the clayey silt are assumed to be as follows:

$c_v = 1.0\ ft^2/day$

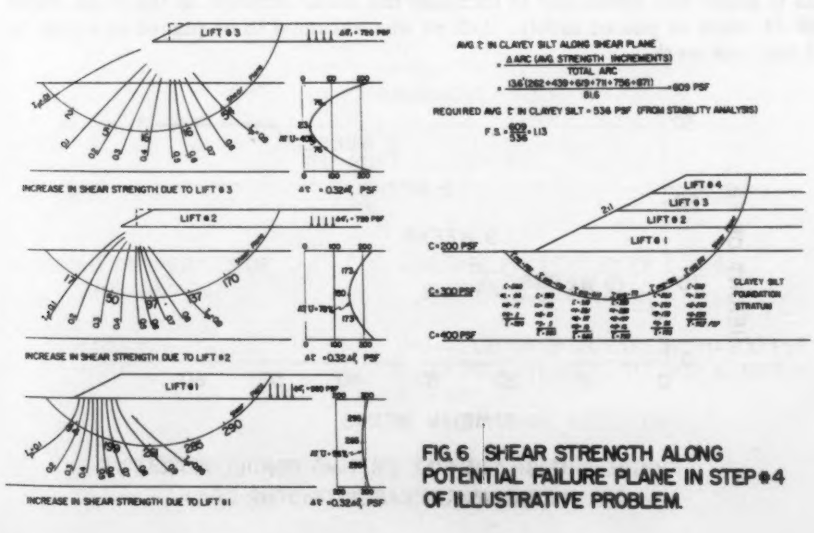


FIG. 6 SHEAR STRENGTH ALONG POTENTIAL FAILURE PLANE IN STEP #4 OF ILLUSTRATIVE PROBLEM.

ϕ_e = 35° as determined from triaxial tests with pore pressure measurements

C = natural strength of clayey silt which increases from 200 psf at the top of the deposit to 400 psf at the bottom.

τ = density of fill = 120 pcf

The problem is to find out how long it will take to construct the embankment and what the approximate loading schedule will be, assuming 2:1 slopes and no berms.

Step #1

The first step in the problem was to find out what height of fill the foundation soil would safely support. A factor of safety of 1.1 was assumed for the analysis. Using Jürgensen's shear stress diagram(5) for terrace loading for the first lift, it was found that a fill of eight feet could be placed immediately. An actual rate of filling four feet per week was assumed. The remainder of the fill was divided into increments as shown in Fig. 2.

Step #2

Next a stability analysis was performed to determine the strength required for the stability of Lift #1 and #2, a total of 14 feet. By trial, it was found that a waiting period of 19 weeks was required for the clayey silt to build up enough strength to permit placing of Lift #2 immediately. Actually, it was assumed that Lift #2 would be placed at a rate of 1.5 feet per week which would allow further consolidation and a somewhat higher safety factor.

Step #3

After the required shear strength was computed to sustain Lift #3 (for a total of 20 feet), it was found that a waiting period of nine weeks after Lift #2 was in place was necessary to increase the shear strength to the point where Lift #3 could be placed safely. Lift #3 was assumed to be placed at a rate of 1.5 feet per week.

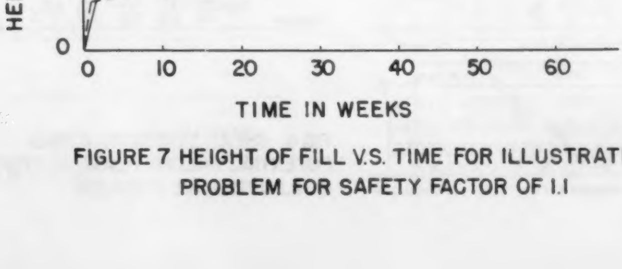


FIGURE 7 HEIGHT OF FILL V.S. TIME FOR ILLUSTRATIVE PROBLEM FOR SAFETY FACTOR OF 1.1

Step #4

By the same procedure it was determined that Lift #4 (making a total height of 26 feet) could be placed following a waiting period of two weeks after Lift #3 was in place. Lift #4 was assumed to be placed at a rate of 1.5 feet per week.

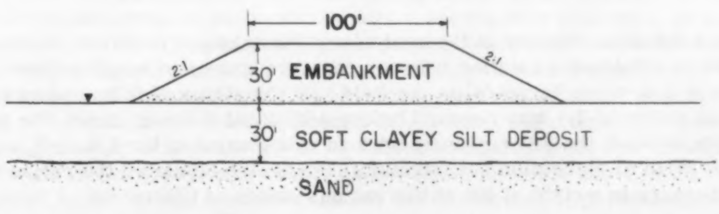
Step #5

After a waiting period of two weeks, following the placing of Lift #4, it was found that the final lift (Lift #5) could be safely placed. A rate of filling of one foot per week was assumed.

Fig. 6 illustrates the increases in shear strength beneath Lifts #1, #2, and #3 in Step #4. These strength increases were summed together with the initial strength and are shown along the potential arc of failure for that stage. The total available strength was then compared with the required shear strength and the factor of safety was computed.

When the stability analyses were performed during the various stages of filling, each lift was assumed to be placed instantaneously. In actual construction operations (except when placed as hydraulic fill), the lifts would not be placed at once. In the example, the placing of fill was assumed to be spread out over a period of time corresponding to actual construction practice where fill is trucked in; therefore the actual safety factor would then be somewhat higher than 1.1. Stability calculations made immediately after the placing of Lifts #2 and #5 indicated safety factors of about 1.2. The safety factor immediately following the placing of the lifts is probably the minimum value that occurs because it would show an increase during the subsequent waiting period.

In order to find out what the maximum safety factor would be under the full height of the embankment after it had fully consolidated over a long period of time, a stability analysis was performed. The safety factor at this state, which represents the maximum safety factor obtainable, was computed to



ASSUMED PROPERTIES OF EMBANKMENT

γ = 120 PSF
ϕ_e = 30°

ASSUMED PROPERTIES OF CLAYEY SILT FOUNDATION SOIL

INITIAL OR NATURAL STRENGTH = 200 PSF AT TOP INCREASING TO 400 PSF AT BOTTOM.
ϕ_e = 35°
C_v = 1.0 SQUARE FEET / DAY

FIGURE 8 EMBANKMENT OVER SOFT FOUNDATION DEPOSIT IN ILLUSTRATIVE EXAMPLE

be 1.39. This indicates that it would have been unrealistic to aim for a safety factor much greater than unity during the filling operation.

The vertical stress distribution beneath the slope of the full height of the fill, determined by summing up the influences of the various lifts along the potential failure arc, was found to agree fairly closely with the distribution beneath the slope according to Jürgenson's terrace loading diagram and to Osterberg's vertical stress distribution values for embankments.(6) This would indicate that the error introduced by assuming strip loading pressure distribution for Lifts #2 through #5 was small. It appears that when the height of an embankment is small compared with the width, as was the case in the example given, Jürgenson's terrace loading vertical stress coefficients do not vary appreciably from the coefficients derived from the Osterberg chart for embankment stresses.

Fig. 7 is a plot of the height of fill against time for the example presented. This graph represents the culmination of the analysis and would be the basis for scheduling filling operation. It is of interest to note that the waiting periods following the placing of each lift can be decreased so that the rate of filling can actually be increased with time. This is due to the cumulative effect of the lifts; the more lifts that are in place, the faster is the consolidation process and concurrent strengthening of the foundation soils. It is believed that the schedule could be deviated from somewhat without affecting the final completion date of filling. A smooth dashed line is drawn through the graph to give an idea of what the average rate of filling might be over various periods of time. Such a plan might be desirable from field construction viewpoint because the job would not have to be shut down entirely between lifts.

Recent work by Schiffman,(7) in which he has extended the theory of consolidation for time-dependent loading and varying permeability, may simplify the process of determining the degree of consolidation under varying loading schedules. In this paper we have dealt with a series of loads placed instantaneously.

Pore Pressures Beneath Full Height of Embankment

From the data obtained in the analysis of the example problem, it was attempted to establish a relation between pore pressure and height of embankment that would be useful in the field. At the states of filling when a factor of safety of 1.1 was reached before additional fill was added, the pore pressure beneath the full embankment load was computed for 1/4, 1/2, and 3/4-the depth of foundation soil stratum. Below are tabulated the ratios of pore pressure to weight of fill at the various states of filling:

Height of fill in feet	Height to which fill could be raised (assuming lift placed immediately and a safety factor of 1.1)	pore pressure ÷ weight of fill at 1/4 and 3/4 depth	pore pressure ÷ weight of fill at 1/2 depth
8	14	.179	.25
14	20	.196	.274
20	26	.292	.398
26	30	.291	.404

The ratio of pore pressure to weight of fill does not appear to show anything more than a general but irregularly increasing trend with raising of the embankment. These values could, however, be used as a rough guide in the field for the case analyzed if there were piezometers installed in the foundation soil stratum under the full height of the embankment. However, it is preferable to have piezometers installed beneath the slope also for stability calculations. Whether the general range of values obtained for the ratio of pore pressure to weight of fill would be applicable for other conditions or not is problematical. It is believed that every case of filling over soft foundation soil is an individual problem in itself, and should be analyzed as such.

Discussion of Assumptions

It is believed that the procedure developed here can be very helpful when dealing with embankments over soft foundation soils. There are, however, several aspects of the problem which deserve further consideration and discussion. The most important of the assumptions made in an analysis of this type is that of the coefficient of consolidation c_v which is determined from the consolidation-time curves. Examination of such curves for many soil types indicates that for a given sample there is generally some scattering in the time at which primary consolidation is essentially complete. This influences the determination of c_v. Therefore, an average value for c_v for a foundation soil stratum should be selected, if possible, from a study of consolidation test results from various depths. Using an average value of c_v is no doubt an oversimplification, but at the present time there does not appear any more rational method for selecting a c_v value.

The theory of consolidation is based on the assumption of a constant permeability, although it is generally recognized that this is not the case in the consolidation process. It is reasonable to assume that as the void ratio of a clay layer subject to loading decreases during the consolidation process, a concurrent decrease in the coefficient of permeability (and coefficient of consolidation) should occur. Zeevaert,[8] for example, has presented curves showing a continuing decrease in permeability with a decrease in void ratio on a sample on which a series of direct permeability tests were performed under increased load increments; the tests were conducted with a standard consolidometer used as a permeameter and measurements were taken after completion of consolidation under each load increment. Unfortunately, however, there is generally scattering in the c_v values computed from the various load increments in many standard consolidation tests, as noted above, which makes it difficult to detect any such decrease in the coefficient of permeability. The recent work by Schiffman[7] has treated the problem of varying permeability in the consolidation process theoretically.

The determination of c_v from the conventional semi-log plot of time-consolidation readings for organic soils is difficult because of the lack of a well defined break between primary and secondary consolidation. More pronounced breaks for organic soils can be obtained from plots of the time-consolidation data on a square root of time scale.

A decrease in permeability and c_v of the foundation soil during placing of an embankment would tend to flatten out the curve denoting height of fill against time (as shown in Fig. 7). A resulting curve through the plot in Fig. 7, for example, might then be more like a straight line rather than an upward turning curve.

As previously noted, there was little error introduced in vertical pressure distribution beneath the embankment slope by assuming strip loading pressure distribution for the fill increments.

Methods of Accelerating Rate of Filling

The most commonly used method of accelerating consolidation of soft foundation soils and expediting the rate of placing embankment material is by the use of sand drains. The problem of predicting the safe rate of filling still remains, however, despite the increased rate of strengthening of the foundation soil. With sand drains, the process of consolidation becomes three-dimensional instead of one-dimensional because of the added factor of radial drainage.

From Terzaghi,[9] the average per cent of consolidation U for a clay layer can be computed from the expression:

$$100 - U\% = (1 - U_r\%)(1 - U_z\%)$$

where $U_r\%$ = per cent radial consolidation

$U_z\%$ = per cent vertical consolidation

The degree of consolidation for any depth in a clay layer can be determined from the one-dimensional process of consolidation, (see Fig. 4). However, the author has no knowledge of a method of computing the per cent consolidation at a given depth in a clay layer for the three-dimensional case where sand drains are involved. Such a value can be approximated by assuming a value between the per cent consolidation at the given depth due to vertical or one-dimensional consolidation and the average per cent consolidation U for the layer due to three-dimensional consolidation with sand drains. This value of per cent consolidation at a given depth in the clay, although admittedly crude, will still enable an approximate solution to be made for such a problem.

The use of flatter slopes or berms, or both, will permit a somewhat faster rate of raising an embankment having minimum slopes because the intensity of the shearing stresses beneath the slopes are decreased. The choice of whether to use flatter slopes and/or berms is a matter of balancing the additional cost of material against a saving in time for construction. A given amount of material will be more effective if used as a berm than on a flattened slope because a berm will act as a greater counterweight against overturning.

Field Control

All of the foregoing analysis and procedures dealing with the prediction of safe rate of raising embankments over soft foundation soils is based upon theoretical relationships, physical properties determined by laboratory tests, and the assumption of uniform conditions which do not always exist. Therefore, a certain amount of averaging and approximating must be done. Because of this, and because of the large number of factors involved in the problem, a predicted rate of filling should be used as a guide and the actual safe rate of filling in the field must be governed by close field observations of pore pressures, settlements, and lateral movements at the toes of the slopes.

The shear strength is a function of the effective stresses. Therefore it is essential to know what the effective stresses are in the foundation soil

throughout the course of construction. Because the pore pressures and resulting effective stresses will vary with depth and location beneath a slope, the Swedish Method of Slices is believed to be the best method for analyzing stability at any time. This requires the installation of piezometers at various depths and locations beneath the embankment slopes so that the pore pressures can be determined on or near the many potential failure planes that will develop during the course of raising the embankment. The variation in shear strength beneath the embankment in Fig. 6 demonstrates the need for piezometers beneath the slope. It is not enough, for a rigorous analysis, to have piezometers installed only below the top and bottom of the slope.

During periodic observations of piezometer readings, field personnel should be on the alert for unexpected increases in pore pressures which may be an indication of incipient failure.

Stability analyses should be performed periodically during construction so that the desired safety factor against slide failures can be maintained. This involves a good deal of repetitious calculations, but it is believed to be the only rational method of accurately controlling the stability and rate of filling of an embankment. The rate of filling on many sand-drain projects in the past has been apparently governed by pore pressure vs. height-of-fill relationships which appear to have little scientific basis and are believed to be simply rule-of-thumb methods based on experience and judgment. Such methods can lead to overly conservative practice.

A double-tube type of piezometer is believed to be preferable to the older, more generally used single-tube type, particularly in organic deposits where marsh gases may be present. A double-tube type of piezometer has two lines leading to the piezometer tip, thus permitting flushing out of accumulated gas bubbles from the entire system. Gases or voids in the piezometer system cause time lags in pore-pressure readings. Double-tube types of piezometers were originally designed and used by the Bureau of Reclamation; similar double-tube types have subsequently been installed in British projects by the British Research Station and in recent sand-drain projects in this country.

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Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

MAJOR POWER STATION FOUNDATION IN BROKEN LIMESTONE[a]

W. F. Swiger,[1] F. ASCE and H. M. Estes[2]

ABSTRACT

Subsurface pinnacles of limestone surrounded by rubble in clay of varying consistency require detailed examination at each building column and equipment foundation to establish an economical type of design for each foundation for a steam power plant.

That surface appearances can be deceiving has long been accepted as a truism of foundation engineering. The site of the Will County Station of the Commonwealth Edison Company, however, went well beyond mere deception.

The station is located about 30 miles southwest of the center of Chicago on a narrow strip of river-bottom land between the Des Plaines River and the Chicago Drainage Canal. The river at this location cuts on a hard crystalline limestone of Silurian age. Rock is visible in the river, which is very shallow, as surface exposures in the site and along the edges of the drainage canal. Despite these apparently ideal conditions, it was found necessary to use a thick mat of reinforced concrete for the first two units. Conditions under Unit 3, which is of 275,000 kw capacity, were so poor that even this type foundation was found unsatisfactory and a radically different design was developed.

Foundation conditions for Units 1 and 2 were investigated by numerous core borings which showed that while rock in most of the area was very close to the surface, there were areas of erosion, cavernous conditions and badly shattered rock. These areas of unsound rock were erratic in size, location and depth.

The design adopted for Units 1 and 2 was a mat, or raft type foundation, having sufficient strength to span the areas of questionable rock, carrying the

Note: Discussion open until March 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2215 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 5, October, 1959.

a. Presented at the October 1958 ASCE Convention in New York, N. Y.

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superimposed loads to areas of sound rock. For these two units, the areas of poor rock were limited, not excessively wide and many of the heaviest loads came on areas of relatively sound rock.

Construction records and photographs for Units 1 and 2, and for the discharge tunnel which passes through the area occupied by Unit 3, together with examination of rock exposure along the drainage canal, indicated that the rock structure under Unit 3 is significantly worse than for Units 1 and 2.

It was decided, in order to make an acceptably accurate analysis of foundation conditions, it would be necessary to develop a clear understanding of the geological history and conditions which caused the peculiar formation underlying the station.

Review of the available core boring data indicated that an adequate analysis could not be based on a series of borings located only from surface exposures. Accordingly, it was decided to strip the limited overburden from the area of the power station. This permitted detailed examination and mapping as a basis for later studies. Areas of sound rock in place were carefully located and mapped in detail as were areas of displaced rock, cavernous or eroded areas and areas of broken rock.

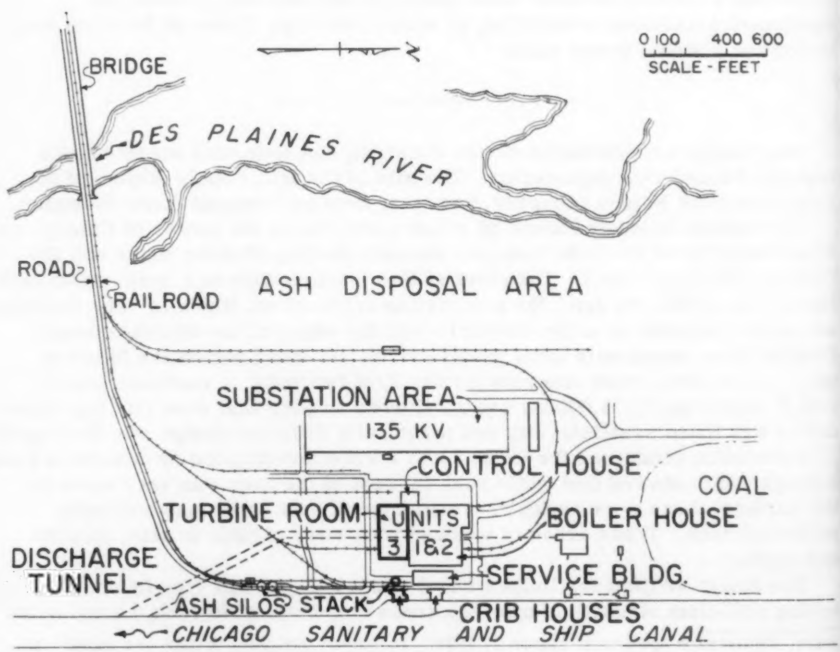

FIG. 1

PLOT PLAN

Geology

Bedrock in this area is the Niagara formation of Silurian age. This formation consists primarily of a series of hard crystalline limestones with many chert inclusions. Colors range from light gray to pink.

At the beginning of Pleistocene Time ground surface in the area was apparently somewhat higher than at present. Probably the area was drained by a river lying to the east and north of the site which occupied a valley several hundred feet in depth. Solution along joint systems from ground waters moving toward the river extended deeply into the limestone. Thus, at the beginning of the Ice Age, Pleistocene Time, the surface of the limestone was crisscrossed by a system of open solution joints and caverns so that the upper 150 ft or more of limestone was essentially a system of individual pinnacles of small areal extent. The general condition at that time was probably as illustrated on Fig. 2.

During the Pleistocene Time, the area was overridden at least three times by continental ice which advanced to a point several hundred miles south and probably had a maximum thickness over the site of several thousand feet. This tremendous weight and the drag it exerted upon the soil crushed the pinnacles and cavern roofs into the openings between and under them, filling these openings with a mass of broken limestone and clay. During Late Wisconsin Time, the most recent glacial period, the Lake Michigan lobe hesitated in its retreat or readvanced to a position slightly south of the present shore of Lake Michigan, creating a large moraine which extends for a number of miles parallel to the lake shore. Drainage from this lobe was primarily along the Des Plaines River which for many thousands of years must have been a raging torrent. This prehistoric river stripped the area of the site bare to bedrock except for an occasional thin mantle of slabby alluvials. It extensively eroded along joints and areas of broken rock where the pinnacles had previously been crushed and displaced by the ice.

Foundation Conditions

Thus, the underground of the site is comprised of a series of pinnacles of limestone extending nearly to the ground surface. These are separated from

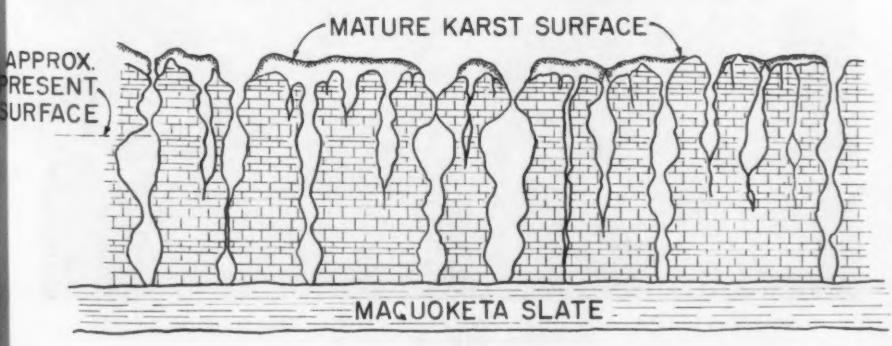

FIG. 2

SECTION REPRESENTING GENERAL GEOLOGY AT SITE

each other by solution channels which are filled with a broken rubble of downward crushed limestone blocks in a matrix of clay of varying consistencies. Some of the clay was preconsolidated to hardpan by the weight of the glacier. At other locations the clays are recent deposits of the Des Plaines River, some of which are above the liquid limit in moisture and very soft in consistency. Fig. 3, a photograph of the wall of the excavation for the discharge tunnel within Unit 3, shows the conditions found with pinnacles of sound rock in place surrounded by areas of crushed and disturbed materials. The solution channels are of varying depths and widths. The deepest extends to the Maquoketa Shale which, being insoluble, limits the depth of solution. The top of shale lies about 130 ft below present ground surface.

The solution channels and the later river erosion generally followed the joint systems. The dominant joint systems runs north about 60 deg east, with secondary systems running north 45 deg west, north and nearly due east.

Detailed geological mapping, to locate all areas of displaced material and sound rock in place at the surface, indicated that the most serious conditions were in the boiler room. This was especially critical because the largest loads in a steam power station occur in the boiler room, where individual column loads are as much as 2,500 tons.

FIG. 3

WALL OF EXCAVATION FOR DISCHARGE TUNNEL UNDER UNIT 3 SHOWING PINNACLE OF SOUND ROCK AT "A" WITH DISPLACED BROKEN MATERIAL TO EITHER SIDE

A general boring program, using a conventional boring spacing independent of concentrated station load conditions, would have contributed little to the knowledge of conditions in the area as it affected foundation design. Borings were considered necessary, however, to determine depth to sound material in place and to check for undercutting of pinnacles. Surface examination indicated the dominant solution channels in the boiler room lay along the north 60 deg east joint system. It was decided to core bore at each column, using two borings straddling the more heavily loaded columns. These borings were located on a line trending north 30 deg west at a point 5 ft either side of each column, since this offered the maximum probability of finding zones of erosion and undercutting. For lighter loads, or where surface examination indicated more favorable conditions, a single boring was used. Boring locations are shown on Fig. 4. The pinnacles of sound rock found in the boiler room area were relatively small with areas of broken displaced material as much as 55 ft wide between. Many of the heavy boiler column loads came over areas of broken rock and support on rock along the east wall, column line F', was almost totally absent.

Conditions appeared much better in the turbine room. Areas of solution or of displaced material were relatively small except along the easterly edge.

The discharge tunnel which was constructed with Units 1 and 2 was provided with shear keys and stub steel at its top so the walls could be built up to form a pair of beams approximately 25 ft in depth. These walls which were

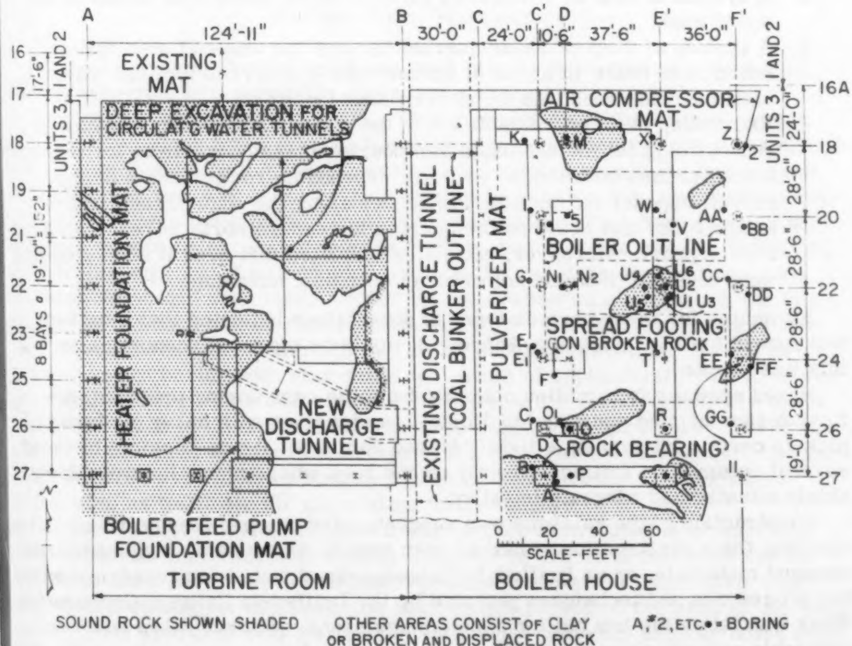

FIG. 4

PLAN AT SURFACE OF ROCK-BOILER HOUSE AND TURBINE ROOM

located under the east column line of the turbine room and under the first line of boiler room columns served to distribute column loads to the areas of sound rock.

The rock underlying the plant, where in position, is a hard crystalline limestone but because it is composed of pinnacles and is of a broken and variable nature, unit load under footings placed near the surface was limited to 15 tons per sq ft.

Deep massive rock at the base of caissons was considered capable of safely carrying 40 tons per sq ft.

Boiler Room Foundations

The tremendous masses of broken rock and boulders which blocked the solution channels would have completely prevented driving of piles of any ordinary type to bearing on sound rock.

Accordingly, other methods of founding were considered. The following comparative studies and estimates were made to permit evaluation of various methods considered feasible for founding the boiler room:

1. A mat construction generally similar to that used for Units 1 and 2 but with increased thickness and reinforcement to span the greater distances of unusable materials found. This mat would be supported directly on the rock pinnacles.
2. A system of heavily reinforced girders which would span areas of unsound rock.
3. A system of deep caissons carried through the unsound materials to sound rock below in areas of broken rock and spread footings where sound surface rock was found at column locations.
4. Excavating to an appreciable depth the materials between dependable rock throughout the extension and pouring mass concrete to form unreinforced arches between the areas of dependable rock. Such a design proved impractical because the arches along the edge of the existing boiler room mat could not be carried far enough north to reach a sound rock support. Moreover, estimates indicated that even if such a design were feasible, it would not offer an economic advantage.

Estimates for these various types of foundations indicated that a caisson and spread footing foundation, Scheme 3, would be the most economical and this was adopted.

Based upon the information obtained from the core boring program, the final design adopted consisted of 15 caissons together with heavy girders to pick up certain intermediate lightly loaded columns. It was possible to found several columns on footings directly on the rock where pinnacles were fortunately situated and adequate in extent.

Construction of the caissons was subcontracted on open competitive bidding allowing their construction either as open type caissons using lagging to hold unsound materials, or as Drilled-In Caissons constructed in accordance with the procedures and techniques patented by the Drilled-In Caisson Corporation. Work was let on the low bid for open type caissons, although there was considerable concern about controlling ground water during construction with this type of caisson. Fortunately, this did not prove to be too difficult.

Caissons vary in size from 3 to 7 ft diam, being proportioned to carry the load imposed upon them. They are reinforced throughout their length and filled with high-strength concrete. Bottoms were belled as necessary to meet the design bearing value of 40 tons per sq ft. They vary in depth from a minimum of 59 ft to a maximum of 132 ft.

Based upon the core borings made at each caisson, a tentative founding elevation was established for each. As the work progressed, each caisson was frequently inspected. Before it was released for concreting, a detailed inspection was made by the Engineers and additional core borings were made at the bottom of each caisson at the locations selected by them, to verify and supplement the other core borings.

Concreting caissons in the dry was impossible because of the large seepage flows into them. Accordingly, each caisson was flooded to static water level, after which the concrete was placed using tremie methods. Several of the caissons were carried into the Maquoketa Shale which is a thinly bedded hard gray shale. While firm and compact in place and an excellent founding material, it deteriorates at the surface when exposed to the air. Therefore, special instructions were given that for all caissons carried to the shale, the bottom and sides where in the shale were to be cleaned down again following final inspection, and the caissons immediately filled with water. Concreting of each caisson was completed as quickly as the caisson could be flooded to static water level.

Turbine Room Foundations

At the surface of the turbine room rock structure there was a deep clay pocket extending in a generally east to west direction through the central portion of the turbine mat location. This extended from the area of severe erosion lying along the discharge tunnel to a point westerly, beyond the end of the turbine foundation. A large block which occupied nearly the entire area of the turbine mat was initially thought to be in position but later examination showed it to be a cavern roof block which had been pushed downward. Since the area was to be excavated to considerable depth for the tunnels underlying the turbine support, it was decided not to make any deep explorations until the excavation had been made. After the excavation was completed, the conditions of the rock are as represented on Fig. 5. Extensive portions of the turbine mat area were underlain by clay at founding elevation. By the extensive use of boring with star drills to a depth of approximately 28 ft, it was proved that there was a sufficient amount of sound rock along both the north and south sides of the turbine foundation to support the structure. The distribution of this rock dictated the design of the foundation which essentially consists of three deep girders spanning from north to south. These girders serve also as walls for the circulating water tunnels.

At the floor of the tunnel the load was further distributed in an east and west direction to reduce unit loading as much as practical and when the design was completed with the turbine, condenser, generator, and other miscellaneous equipment installed, the total weight placed on the area at the foundation level averaged only about 5,000 psf, which was only slightly more than the weight of the rock removed during excavation for the turbine foundation and tunnel structure.

Foundation conditions at the boiler feed pump area were poor so that it was found necessary to install a mat which spans from sound rock on one end to the circulating water tunnel at the other, to support these pumps. This mat also supports several of the turbine room stub columns.

Star drill holes for exploring rock were extensively used for wall and other footings in the turbine room. This proved the desirability of using continuous footings along the west wall of the turbine room to span over small ravines and uncercut areas. In general, the bearing load on the upper structure of rock in the turbine room was limited to 15 tons per sq ft, as in the boiler room area.

Ground Floor Slab

The ground floor slab was supported upon a fill of crusher run rock grading from dust to about 3/4 in. size. This material was placed and compacted in thin lifts. Ten-ton rollers were used for compaction, except in restricted areas where hand-operated pneumatic tampers were used.

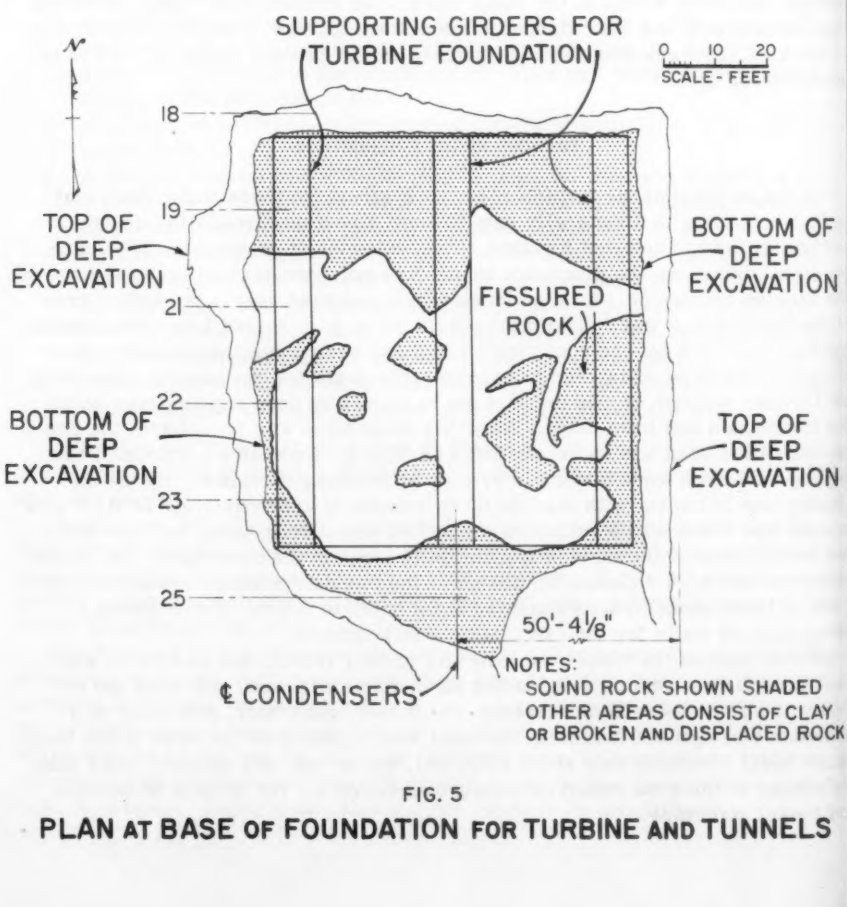

FIG. 5

PLAN AT BASE OF FOUNDATION FOR TURBINE AND TUNNELS

Stack

The concrete chimney, which extends to a height of 450 ft above ground grade, was originally planned to be placed upon an octagonal mat about 48 ft across the flats. The rock at the surface in this area was fairly sound but it was necessary to remove it to appreciable depth because of excavation for the adjacent screen well and for circulating water piping. It was decided to do no exploration work until the deep excavation had been completed. When this excavation was completed to the depth required, limited clay areas were disclosed. The size and direction of these clay pockets are represented on Fig. 6, and, after exploration with star drills, the location of sound rock was such as to dictate the use of a square mat reinforced especially to span the uncertain areas.

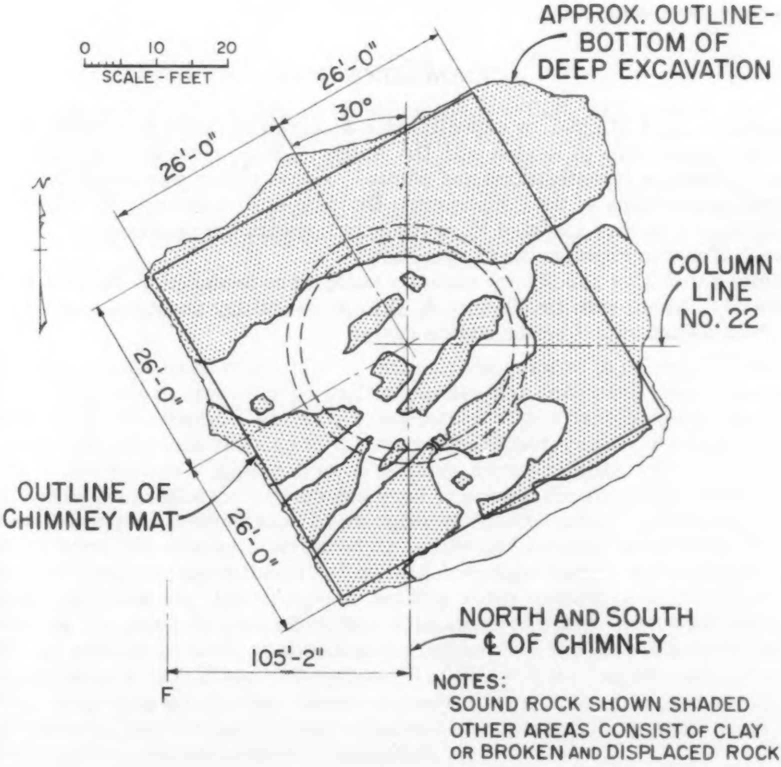

FIG. 6

PLAN AT BASE OF FOUNDATION FOR CHIMNEY

Screen Well

Foundation conditions in the screen well were generally good. There were a few pockets of broken rock and clay at the founding grade, but these were easily spanned by the slab forming the bottom of the screen well and no special precautions or provisions were necessary.

CONCLUSION

In spite of the discouraging conditions found under the site, a safe and economical foundation was developed by use of caissons to sound rock where required and of concrete footings and mats in localized areas where sufficient rock was found in place to support the loads imposed. To utilize the existing rock in the most economical manner required very detailed study on almost a column by column basis.

ACKNOWLEDGMENTS

Unit No. 3, of 275,000 kw capacity, was designed by Stone & Webster Engineering Corporation of Boston with Dr. Ralph B. Peck serving as consultant in the foundation investigations and studies. Construction was under the general supervision of the Construction Division of Commonwealth Edison Company of Chicago, assisted by continuous inspection by members of Stone & Webster Engineering Corporation.

Units 1 and 2, of 150,000 kw capacity each, were designed by Sargent and Lundy of Chicago, with Dr. George H. Otto as consulting geologist and Dr. Peck as consultant on foundation design.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

STRUCTURE AND STRENGTH CHARACTERISTICS OF COMPACTED CLAYS

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SYNOPSIS

The influence of soil structure on shrinkage, swelling, swell-pressures, stress-deformation characteristics, undrained strength, pore-water pressures and effective strength characteristics is described and examples of the relationships between composition and the 'as-compacted' strength of compacted clays are presented. The influence of shear strain on soil structure is demonstrated and used to explain the effect of various methods of compaction on soil strength characteristics.

Convincing evidence of the type of structure developed in compacted clays and the influence of structure on soil properties has been presented in recent papers by T. W. Lambe.(1,2) As a consequence of this development, many previous observations relating to the strength characteristics of compacted clays, which formerly appeared to be isolated pieces of information, may now be fitted into a consistent pattern and used to predict the probable behavior of the various types of compacted clays under different loading conditions. Furthermore, the concepts advanced by Lambe can readily be extended and modified to explain the influence of method of compaction on soil structure and strength, and the significance of molding water content as a factor determining the strength characteristics of compacted clays. In the following pages an attempt is made to demonstrate and explain the significance of these various factors in relation to the strength of clays in the "as compacted" condition. In a companion paper the same concepts are utilized to provide an explanation of the strength characteristics of compacted clays which have been soaked to saturation after compaction.

Note: Discussion open until March 1, 1960. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2216 is part of the copyrighted Journal of the Soil Mechanics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 5, October, 1959.

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Structure of Compacted Clay

The arrangement of the clay particles in a compacted soil, as originally conceived by Lambe,[1] is illustrated in Fig. 1(a). The changes in arrangement at different stages of the density-water content relationship are explained as follows: At point A the small amount of water present results in a high concentration of electrolyte which prevents the diffuse double layer of ions surrounding each clay particle from developing fully. The double layer depression leads to low inter-particle repulsion, resulting in a tendency towards flocculation of the colloids and a consequent low degree of clay particle orientation in the compacted soil. This type of structure has been termed a "flocculated" arrangement of soil particles. If the water content is increased to point B, the electrolyte concentration is reduced, resulting in an expansion of the double layer, increased repulsion between particles and a low degree of flocculation; that is, an increased degree of particle orientation. Further increase in water content at point C increases this effect and results in a still greater increase in particle orientation.

A system of parallel particles, which is approached at point C, has been termed a "dispersed" system. Thus, in general it may be stated that compaction of a clay soil "dry of optimum" tends to produce a flocculated arrangement of particles, while compaction of the same soil "wet of optimum" tends to produce a dispersed arrangement of particles.

Evidence in support of this concept, obtained by Pacey[3] using optical techniques developed by Mitchell,[4] is shown in Fig. 1(b), and similar data obtained from compacted samples of kaolin clay is presented in Fig. 2. It should be noted that the samples on which these observations were made were compacted by kneading in the Harvard miniature compaction apparatus[5] and that there is no direct confirmation of the above hypothesis for samples prepared by other methods of compaction. It should also be noted that the changes in structure described above are not likely to develop in all compacted clays. It is not difficult to visualize that in some soils the tendency of the particles to flocculate will be so great that the small changes in water content in going from dry to wet of optimum will not produce any significant change in the degree of flocculation of the compacted samples. And again, some soils may tend to disperse at water contents dry of optimum, in which case they will remain dispersed at water contents above optimum.

Effect of Structure on Soil Properties

Differences in structure resulting from compaction can have a pronounced effect on the engineering properties of a soil (see Lambe, reference 2). Since these effects can be used as indicators of structure, a brief description of their influence on some of the more easily measured soil properties is presented below. It should be noted that all the data presented in this section were obtained from samples prepared by kneading and impact compaction methods, which are quite similar in their effects and for which test data are available to substantiate the changes in structure previously described.

Influence of Structure on Shrinkage

It was observed by Lambe[1] that samples compacted dry of optimum shrink appreciably less than samples compacted wet of optimum, a fact which led to

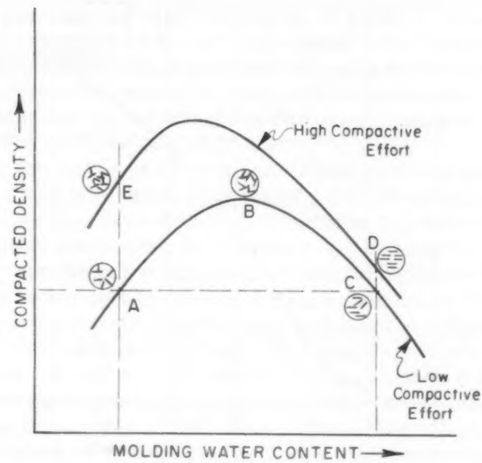

Fig.1(a)-EFFECT OF COMPACTION ON SOIL STRUCTURE.

(After T.W. Lambe)

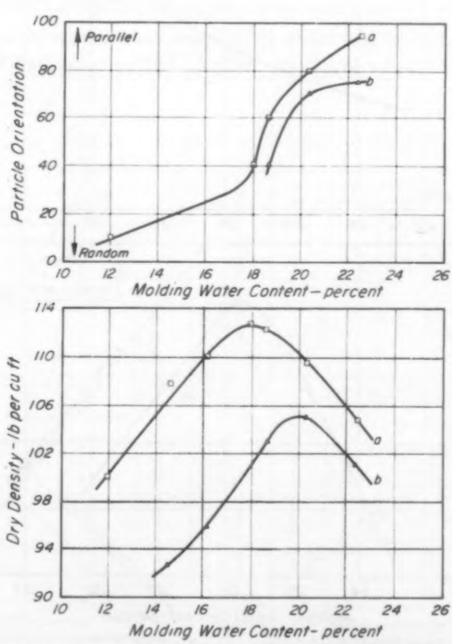

Fig.1(b)-INFLUENCE OF MOLDING WATER CONTENT ON PARTICLE ORIENTATION FOR COMPACTED SAMPLES OF BOSTON BLUE CLAY.

(After J G Pacey, Jr)

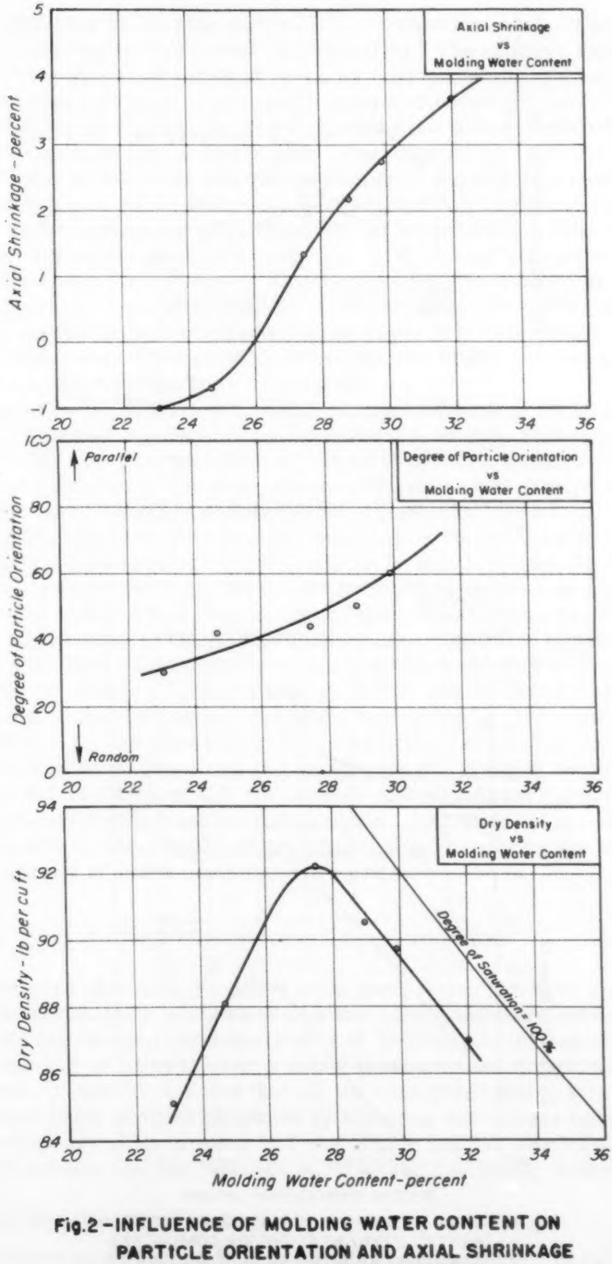

Fig. 2 - INFLUENCE OF MOLDING WATER CONTENT ON PARTICLE ORIENTATION AND AXIAL SHRINKAGE FOR COMPACTED SAMPLES OF KAOLINITE.

the suggestion that shrinkage might possibly be used as a measure of particle orientation. Typical data illustrating this result are shown in Figs. 2 and 3. Fig. 2 presents test data showing the degree of particle orientation and the axial shrinkage of samples of kaolin clay compacted at various water contents. It will be seen that both the degree of particle orientation and the axial shrinkage increase as the water content at compaction increases. Fig. 3(a) presents similar data showing the relationship between compacted condition and axial shrinkage for samples of a silty clay.

Unfortunately the influence of structure alone on shrinkage is not made apparent by data of this type since samples of equal densities compacted wet and dry of optimum have both different structures and different water contents. However, this effect can readily be clarified by allowing the compacted samples to soak at constant volume before measuring shrinkage. In this way two samples can be brought to the same final condition of density and water content but with different structures, thus enabling the influence of structure on shrinkage to be readily determined. The results of tests of this type are shown in Fig. 3(b). It will be seen that samples compacted dry of optimum (and therefore having essentially flocculated structures) exhibited considerably less shrinkage than samples of the same composition compacted wet of optimum. In fact, reference to Fig. 3(a) indicates that soaking at constant volume, at least for this soil, has apparently very little influence on the shrinkage of compacted specimens.

Influence of Structure on Swelling

There is also a considerable amount of evidence to indicate that samples compacted dry of optimum exhibit higher swelling characteristics and swell

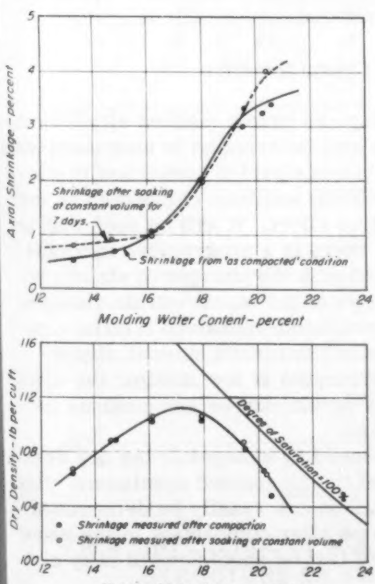

Fig. 3(a)—INFLUENCE OF SOIL STRUCTURE ON SHRINKAGE OF SILTY CLAY.

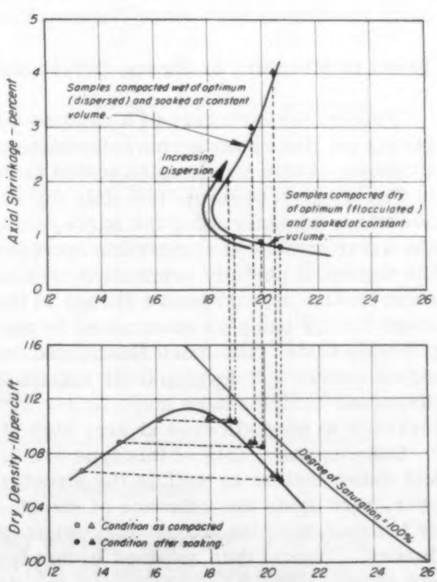

Fig. 3(b)—INFLUENCE OF SOIL STRUCTURE ON SHRINKAGE OF SILTY CLAY.

to higher water contents than do samples of the same density compacted wet of optimum. Thus the increased swell might be interpreted as a manifestation of the greater swelling tendency of flocculated structures than of dispersed structures. Typical evidence of this effect is illustrated in Fig. 4, which shows the final water contents after swelling for samples of sandy clay compacted at different water contents. Some of the water which enters a compacted specimen during swelling is required simply to fill the voids and bring the soil to a saturated condition, as distinct from water absorbed by the soil during the swelling process. Thus samples compacted wet and dry of optimum (and having different structures) will show different increases in water content due to both of these effects. In Fig. 4 the total increase in water content during swelling has been separated into that water content increase required to cause saturation and that due to swelling after the saturated condition has been attained. The marked increase in swelling tendencies of dry side compacted samples (and presumably, therefore, of flocculated structures) is readily apparent.

Influence of Structure on Swell Pressure

Since soil structure appears to be a factor influencing the swelling characteristics of soils, it might also be expected to have a significant influence on the swell pressure exerted by compacted clay. That this is in fact the case is illustrated by the test data in Fig. 5. Compacted samples confined at approximately constant volume by means of the compaction molds and pistons on their upper surfaces were given access to water and the swell pressures exerted on the pistons were recorded. Samples compacted dry of optimum (tending to have more flocculated structures) exhibited greater swell pressures than samples of the same final composition compacted wet of optimum (tending to have more dispersed structures).

Effect of Structure on Stress-Deformation Characteristics

Finally, the structure of a compacted clay can have a marked effect on the stress vs. deformation characteristics of a soil determined in undrained tests. Evidence of this for samples tested in the "as-compacted" condition is shown in Fig. 6 which presents test data for samples of kaolinite compacted at various water contents using the same compactive effort. It will be seen that as the water content at compaction increases, there is a progressive increase in the degree of particle orientation; associated with the changes in structure there is also a progressive change in the form of the stress-strain relationships for the samples determined by unconsolidated-undrained triaxial compression tests. The more flocculated samples have much steeper stress-strain curves and develop their maximum strengths at low strains; the more dispersed samples have much flatter stress-strain curves and continue to increase in strength even at very high strains.

Unfortunately, data of this type are influenced by changes in the dry density and water content as well as the structure of the compacted specimens. However, here again the influence of structure alone can readily be determined by tests on samples soaked at constant volume after compaction to the same density. Typical data obtained in this type of test on specimens of silty clay are shown in Fig. 7.

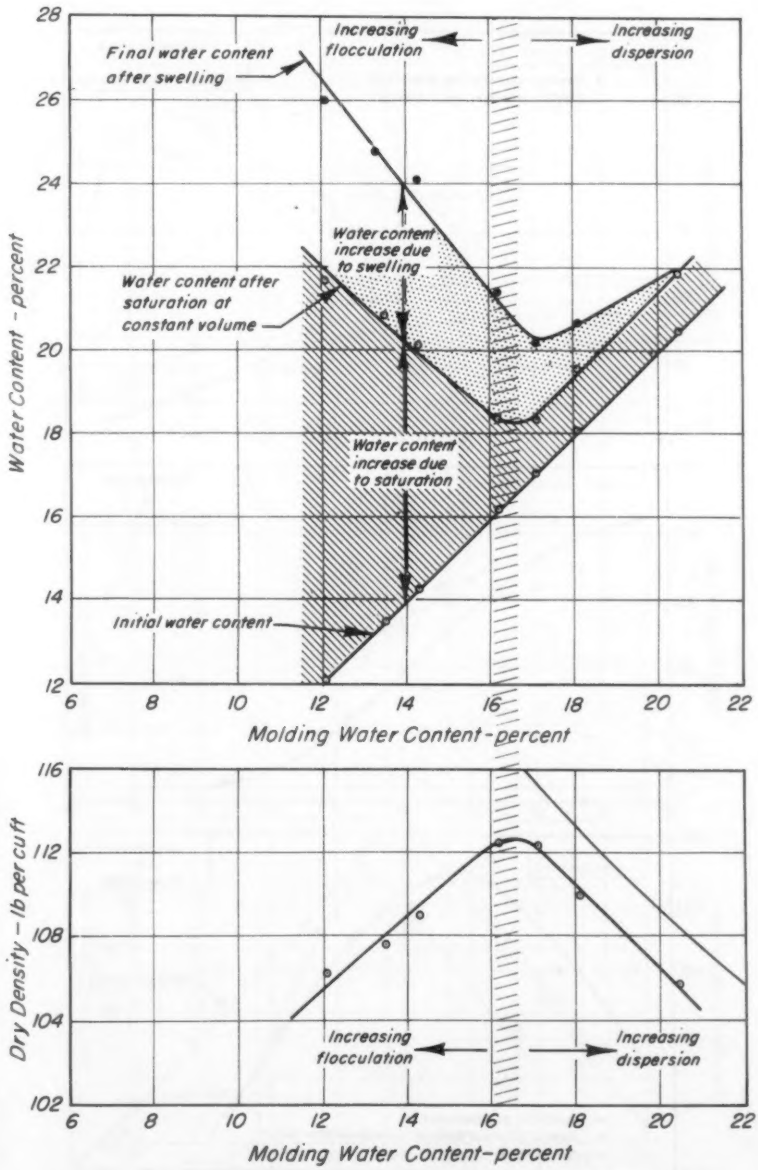

Fig.4 - INFLUENCE OF MOLDING WATER CONTENT AND SOIL STRUCTURE ON SWELLING CHARACTERISTICS OF SANDY CLAY.

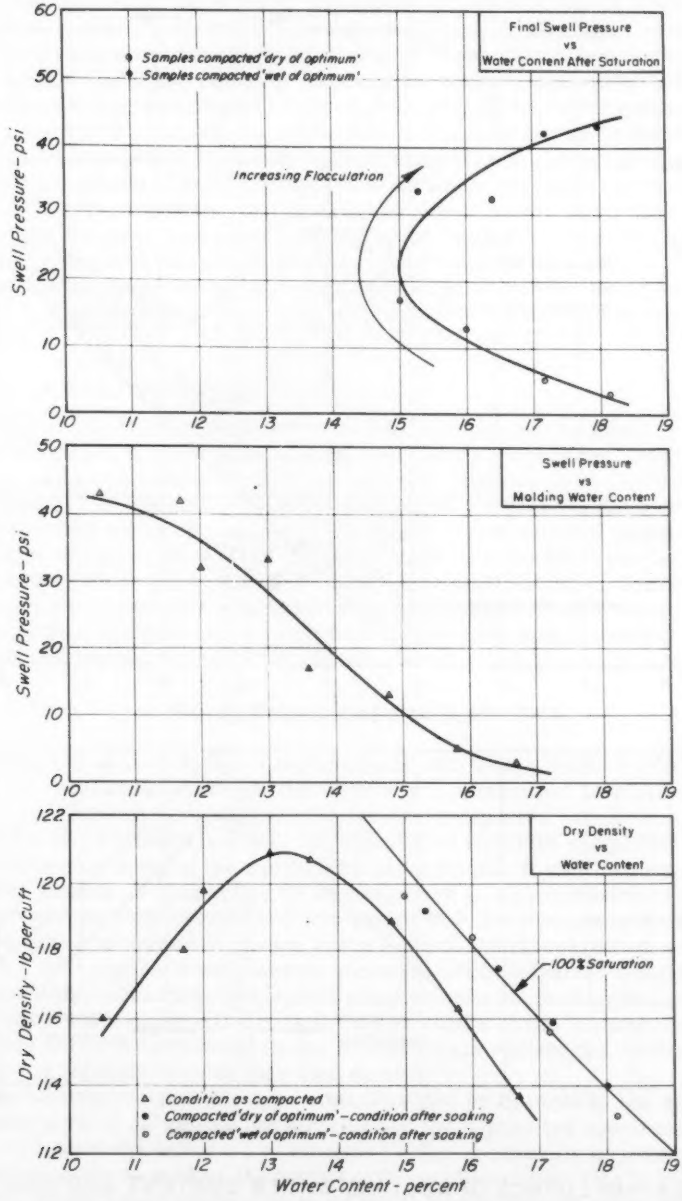

Fig. 5 - INFLUENCE OF MOLDING WATER CONTENT AND SOIL STRUCTURE ON SWELL PRESSURE OF SANDY CLAY.

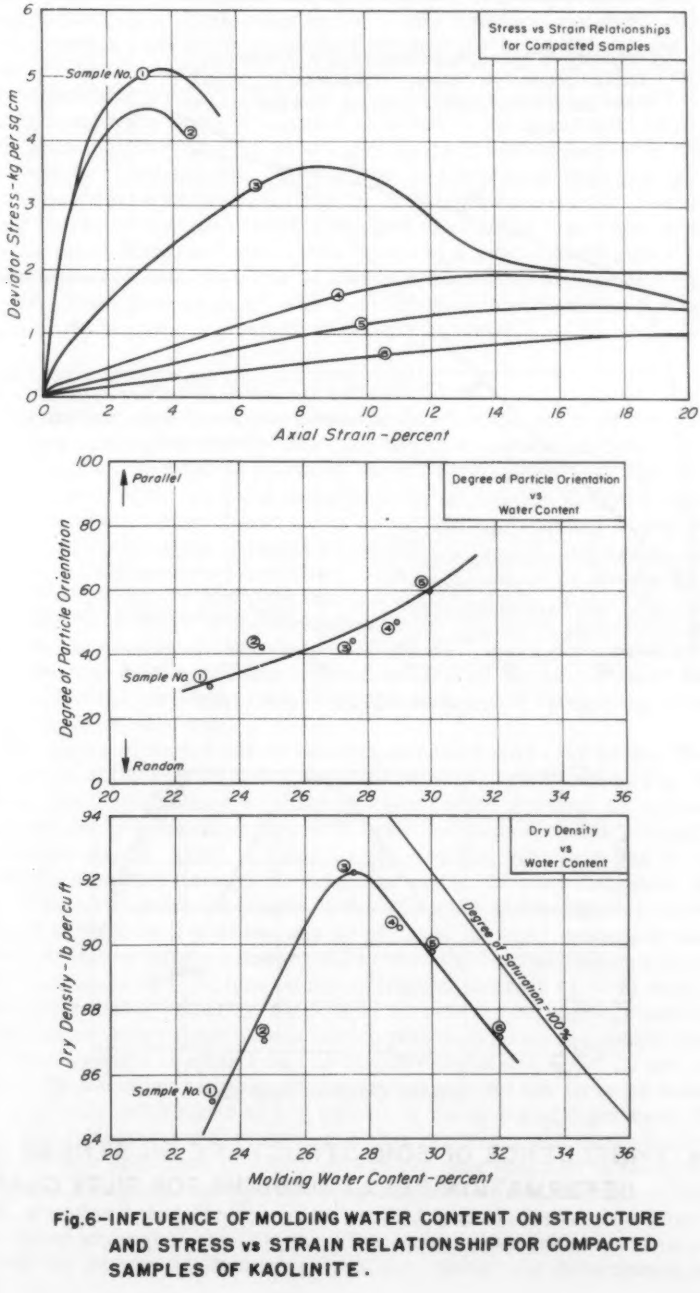

Fig.6-INFLUENCE OF MOLDING WATER CONTENT ON STRUCTURE AND STRESS vs STRAIN RELATIONSHIP FOR COMPACTED SAMPLES OF KAOLINITE.

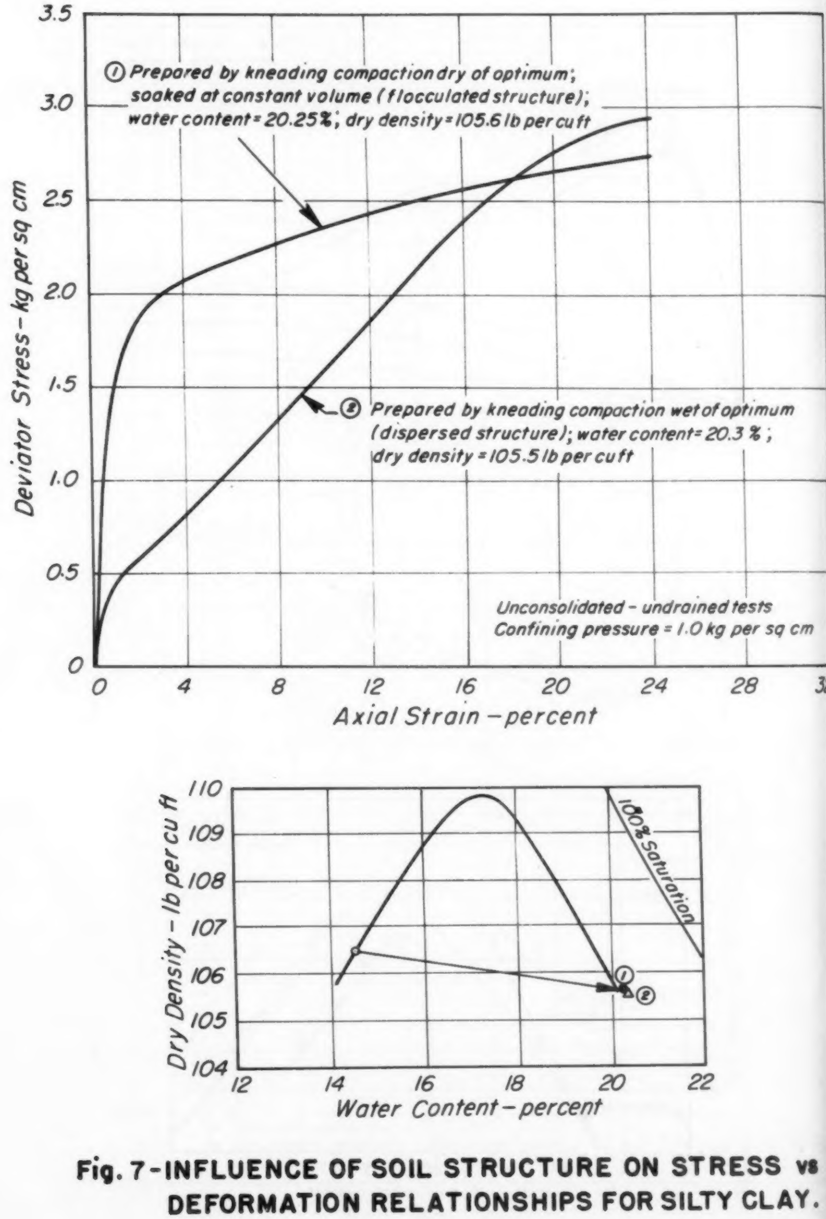

Fig. 7 - INFLUENCE OF SOIL STRUCTURE ON STRESS vs DEFORMATION RELATIONSHIPS FOR SILTY CLAY.

Two specimens were compacted to the same density, one dry of optimum, to produce a flocculated structure, and one wet of optimum to produce a more dispersed structure. Both samples were then soaked at constant volume for 6 days, at which time the samples had attained the same density and water content, close to saturation, but had retained their original structures. The stress-deformation relationships of the samples in undrained triaxial compression tests were then determined as shown in the upper part of Fig. 7. It will be seen that the different structures again produced markedly different stress-strain relationships. It would appear from these data that the more flocculated structures produce an initially steep stress-strain curve with only a slight increase in stress being developed after about 5 per cent strain; however, the more dispersed structures result in a much flatter curve with a consistent increase in resistance to deformation up to strains as high as 25 per cent. The influence of structure on the strain developed at a given proportion of the maximum strength is readily apparent.

Influence of Structure on Undrained Strength

The question must now be considered—what is the effect of structure on the strength of compacted clays? Before this can be answered, however, it is necessary to define what is meant by the term "strength". In recent years the development of equipment and techniques for measuring the pore water pressures induced in soils has led to an increasing tendency to express soil strength characteristics in terms of effective stresses, and some engineers consider any strength determination, without suitable corrections for the influence of the pore water pressures, as of relatively little value. On the other hand, if strength is considered to be the maximum stress which a material, in its existing condition, can withstand, then the pore water pressures induced during loading become a characteristic of the initial condition of the material and the strength of a soil can be determined directly by an undrained triaxial compression test.

The results of such tests on two specimens of silty clay having the same density and water content but different structures are shown in Fig. 7. It will be seen that both specimens, although having quite different structures and stress-strain relationships, reached about the same ultimate strength at about 25 per cent strain. Thus, it might be claimed that structure has practically no influence on soil strength in undrained tests. On the other hand, this conclusion would be quite erroneous if the strengths of the samples were compared at some limiting strain, say 10 per cent, at which stage the more dispersed structure is appreciably weaker than the flocculated structure.

The influence of structure on the undrained strength of soils thus depends on the deformation criterion adopted as a basis for strength determination. In compression tests on specimens having plastic type stress-strain characteristics the strength is often taken as the stress causing about 20 per cent strain. On the other hand, in pavement design tests the strength index of a soil is usually determined at low strains of the order of 5 per cent. Thus, engineers concerned with pavement design problems are likely to conclude, from the data presented in Fig. 7 or similar data obtained in pavement design tests, that structure has a pronounced influence on the strength of compacted soils, with dispersed arrangements producing much lower strengths than flocculated arrangements. On the other hand, engineers concerned with testing soil for foundation studies or earth dam design and determining strengths

at large strains (about 20 per cent), if the specimen has not yet reached its maximum resistance at that point, are likely to conclude that structure has little or no influence on soil strength. These apparently conflicting conclusions can readily be reconciled by a consideration of the influence of structure on the entire stress-strain relationship for a compacted clay.

Influence of Structure on Pore-Water Pressures

The significance of pore-water pressure as a factor affecting the strength characteristics of soils has long been recognized. Since the structure of a compacted clay can apparently have a large effect on the stress-strain characteristics determined by undrained compression tests, it becomes pertinent to determine whether the different resistances to deformation exhibited by specimens having different structures are due primarily to differences in the pore-water pressures developed in the samples or to some inherent characteristic of the soil structure itself.

Consideration of the particle arrangements in dispersed and flocculated structures would indicate that the braced-box type of arrangement of particles in the flocculated system would be more resistant to applied pressures than the parallel particle system of a dispersed structure and consequently would tend to prevent pore-water pressures developing in this type of structure to the same extent that they would develop in a dispersed structure.

Experimental evidence of this type of behavior is provided by the test data in Fig. 8 which shows the stress-deformation relationships and pore-water pressures developed during undrained tests on samples of silty clay having the same density and water content but different structures, as indicated by the forms of their stress-deformation curves. It is apparent that during the early stages of the test particularly, the pore-water pressures in the sample compacted wet of optimum (dispersed structure) are considerably greater than those in the sample compacted dry of optimum (flocculated structure) even though both samples had the same water content during the test. This type of behavior has been observed in a number of tests and appears to be generally characteristic of the influence of structure on pore-water pressures. However, it is interesting to note that at high strains, both samples in Fig. 8 exhibited approximately the same pore-water pressures and approximately the same strength.

Influence of Structure on Soil Strength Characteristics in Terms of Effective Pressures

Finally there remains to be clarified the influence of soil structure on the strength characteristics of compacted clays. Realistic evaluation of soil strength characteristics (as distinct from soil strength) can only be made in terms of the effective stresses acting on a sample at failure. Such stresses can readily be evaluated for the two specimens, having different structures, for which test data is presented in Fig. 8. By subtracting the measured pore-water pressures from the applied stresses the effective or inter-granular stresses acting on the specimens throughout the tests can be determined; and the ratio of the effective major principal stress to the effective minor principal stress with increasing strain can then be utilized as an indicator of the strength mobilizing characteristics of each specimen.

The values of this ratio throughout the tests on the two specimens having different structures and exhibiting quite different stress-strain curves in

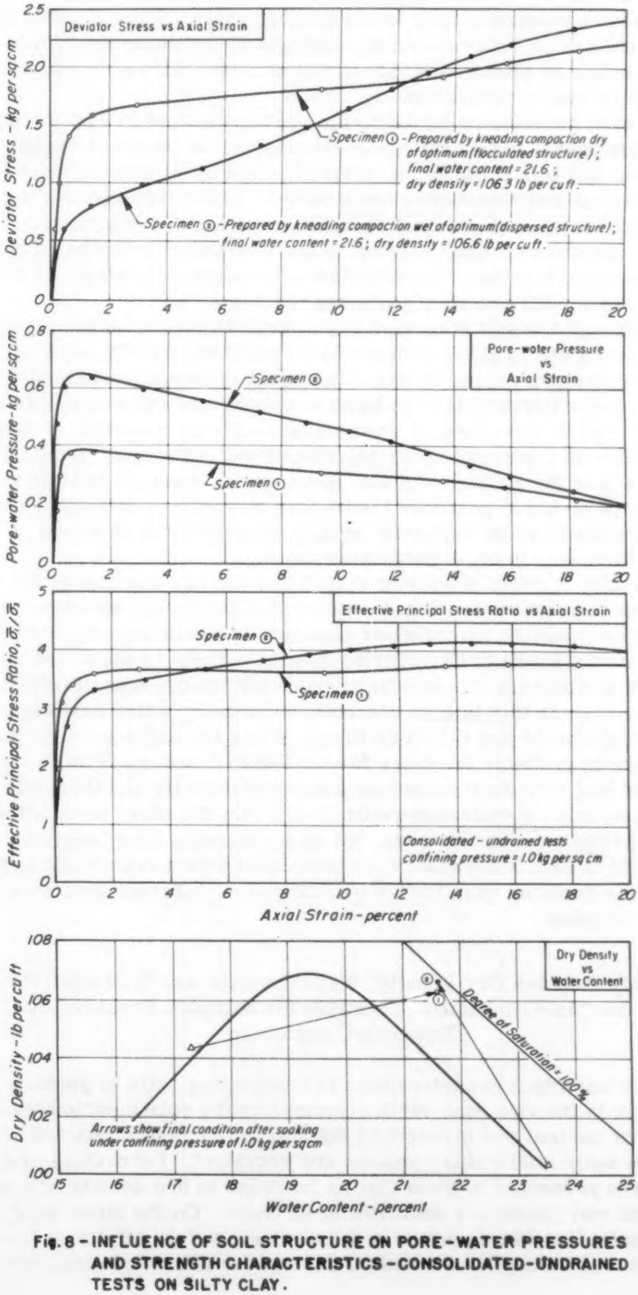

Fig. 8 - INFLUENCE OF SOIL STRUCTURE ON PORE-WATER PRESSURES AND STRENGTH CHARACTERISTICS - CONSOLIDATED-UNDRAINED TESTS ON SILTY CLAY.

undrained tests are also shown in Fig. 8. It will be seen that when the strength characteristics are evaluated in terms of effective stresses in this way the two specimens exhibited almost identical properties. Thus it would appear that structure has no influence on the strength characteristics of this soil expressed in terms of effective stresses and with due allowance made for the effects of pore-water pressures.

Similar data for tests on kaolinite clay are presented in Fig. 9. Here again two specimens having the same composition in terms of density and water content but with different structures (see Fig. 2) exhibited quite different stress-strain relationships when loaded to failure in undrained tests. However, the two specimens also show different pore pressures during the tests and if the strength characteristics are expressed in terms of the effective stresses the rates of mobilization of strength with strain in the two specimens are for all practical purposes the same.

Thus, although the soil structure has a large influence on the pore-water pressures developed in these compacted clays, it apparently has little or no effect on the strength characteristics (in terms of effective stresses) of the clay itself. Since the two clays utilized in these tests exhibited quite pronounced changes in structure, it seems reasonable to conclude, at least tentatively, that this is a general characteristic of soil structure. It would necessarily follow that the influence of soil structure on strength is limited to its influence on pore-water pressures—and thus strength variations due to differences in structure can be explained equally satisfactorily in terms of corresponding differences in pore-water pressures.

For example, it might simply be stated that the two specimens for which strength data is presented in Fig. 8 exhibited different stress-strain relationships simply because of the different pore-water pressures developed in the specimens during loading. It would be equally correct to state that the specimens exhibited different stress-strain relationships because of differences in soil structure (since this in turn determines the pore-water pressures). In effect both statements say the same thing. While the explanation in terms of soil structure is probably the more fundamental, it suffers from the severe disadvantage that structure cannot be measured directly and therefore its influences can only be evaluated qualitatively. On the other hand, although pore water pressure might be regarded as a somewhat less fundamental soil characteristic it can, if necessary, be measured with a reasonable degree of accuracy, thus enabling quantitative evaluations of different specimen characteristics to be made.

Relationship Between Dry Density, Water Content and Undrained Strength in the "As-Compacted" Condition for Samples Prepared by Kneading Compaction

One of the important considerations in compacting soils to produce maximum strength in the resulting earth structure is the relationship between dry density, water content and strength of the soil in the "as-compacted" condition. Here again conflicting opinions are prevalent. Laboratory and field data have been presented to show that an increase in dry density at a given water content may produce a decrease in strength. On the other hand, many engineers insist that this is not so and that increased density must inevitably lead to increased strength. A consideration of the effects of structure on the

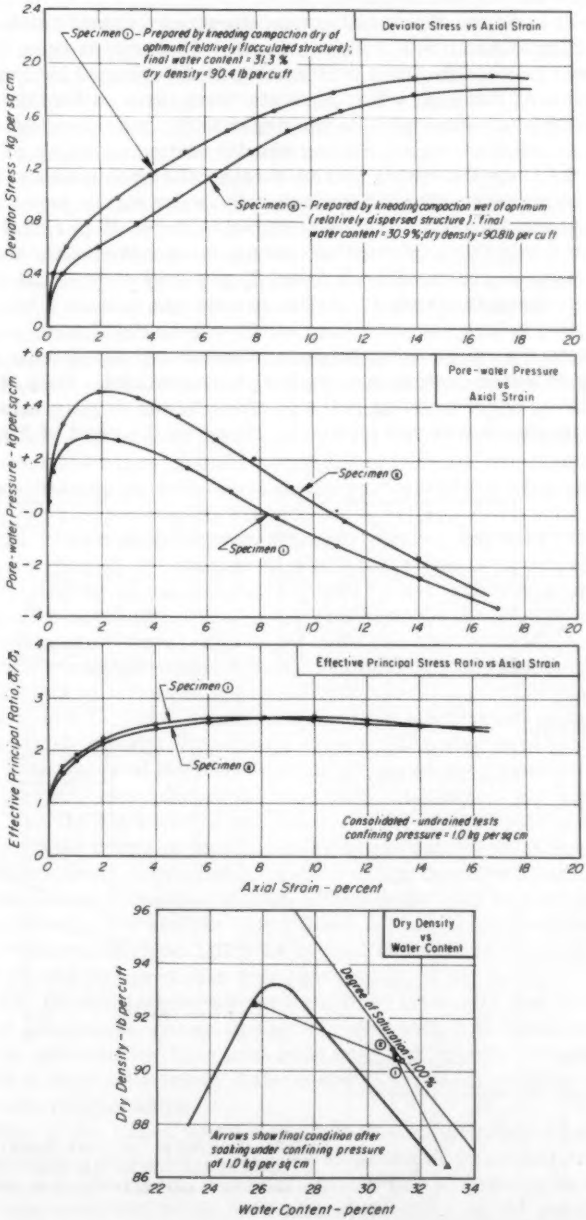

Fig. 9 - INFLUENCE OF SOIL STRUCTURE ON PORE-WATER PRESSURES AND STRENGTH CHARACTERISTICS-CONSOLIDATED-UNDRAINED TESTS ON KAOLINITE.

stress-strain relationships of compacted soils in undrained tests can readily throw some light on this controversial point.

Consider, for example, the density-strength-water content relationship for the silty clay (Liquid Limit = 37, Plastic Limit = 23 per cent finer than .002 mm. = 24) shown in Fig. 10. This relationship is determined by compacting series of samples at different water contents using three different compactive efforts. For each compacted sample the dry density, water content and undrained strength are determined and the results plotted as shown on the left of Fig. 10. In the tests the entire stress-strain relationship was recorded for each sample and the 'strength' determined both as the stress required to cause 5 per cent strain and the stress required to cause 20 per cent strain. It should be noted that these 'strengths' are not simply the points on the stress-strain curve corresponding to 5 and 20 per cent strains but will often be the maximum deviator stress which the sample can sustain if this stress has to be exceeded before the necessary strain can be developed.

From this data the relationship between strength and dry density at various constant values of water content can readily be determined. Thus at any given water content three corresponding pairs of strength and density values can be interpolated from the curves and plotted as shown on the right of Fig. 10.

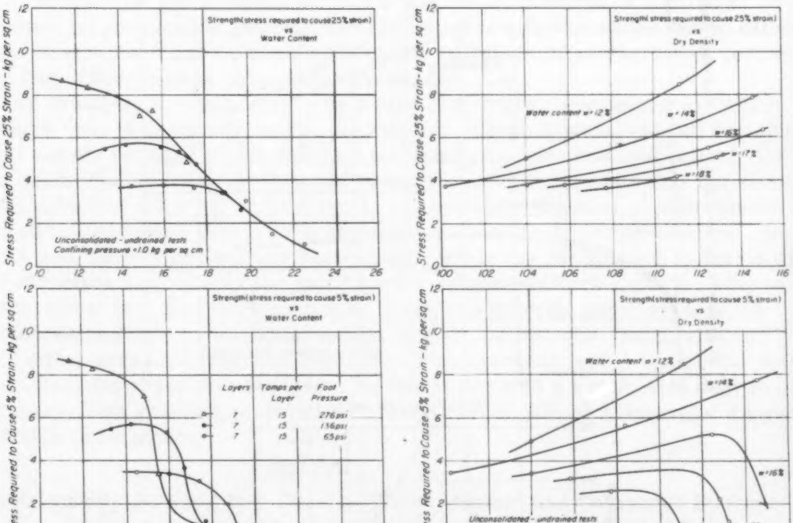

Fig.10-RELATIONSHIP BETWEEN DRY DENSITY, WATER CONTENT AND STRENGTH AS COMPACTED FOR SAMPLES OF SILTY CLAY-KNEADING COMPACTION.

It will be seen that when the 'strength' is determined at low strains, say 5 per cent, there is a marked decrease in strength as the density is increased beyond a certain stage. Reference to the compaction curves shows that these reductions in strength are developed at densities and water contents corresponding to high degrees of saturation or to compacted conditions which are "wet of optimum" on a compaction curve.(7,8) Thus the strength reductions would appear to be associated with the more dispersed structures (and correspondingly higher pore-water pressures) of samples compacted wet of optimum. These data might therefore be interpreted as indicating that as long as structure remains essentially the same, strength will increase with density (the first parts of the curves on the right of Fig. 10) but when marked changes in structure are also incorporated in the data, the strength at low strains may in fact be markedly reduced, in spite of density increases, as a result of the predominating influence of the higher pore-water pressures associated with the dispersed structures.

However, if strength is determined at high strains, samples of this soil having the same composition exhibit approximately equal strengths whether the structure is flocculated or dispersed (see Figs. 7 and 8). Consequently, the soil behaves from a strength point of view as if the structures of all samples were the same, and the relationship between dry density and strength for a given molding water content shows no decrease in strength with increase in density, as may be seen in Fig. 10.

Thus the relationship between strength, density and water content may vary greatly depending on the manner in which the strength is determined, and this in turn will depend on the purpose for which the relationship is being used.

It should be noted that the above considerations do not apply in their entirety to all compacted clays. For some soils the inter-particle forces may be so strong that the changes resulting from compaction at various water contents are insufficient to influence appreciably the tendency of the particles to flocculate (or disperse). Thus, samples compacted wet and dry of optimum will have essentially similar structures. Again, it is also possible that in some soils the structures of the clay fraction of samples compacted wet of optimum are considerably more dispersed than those of samples compacted dry of optimum, but the influence of the difference in structure is masked by other factors (such as a high proportion of granular particles) which produce a steep stress-strain relationship in spite of the dispersed particle arrangement in the clay fraction; in other words, although the clay in a sample has a dispersed structure, the sample may behave from a stress-strain point of view almost as if it were flocculated. A typical example of this type of behavior is provided by the stress-strain relationships for a sandy clay soil (Liquid Limit = 35, Plastic Limit = 19 per cent finer than .002 mm. = 24) from Pittsburg, California, shown in Fig. 11. For this soil, samples compacted wet and dry of optimum and then soaked at constant volume to the same final composition show only minor differences in the form of their stress-deformation relationships.

In either of the cases mentioned above the significance of structure as a factor inducing variations in pore-water pressures and soil strength will be very low. It might therefore be anticipated that for strengths determined at high strains structure would have almost no effect on the results and, at constant water content, an increase in density would always result in increase in strength. Similarly, for 'strength' values determined at low strains, the minor changes in structure or in stress-strain characteristics would have

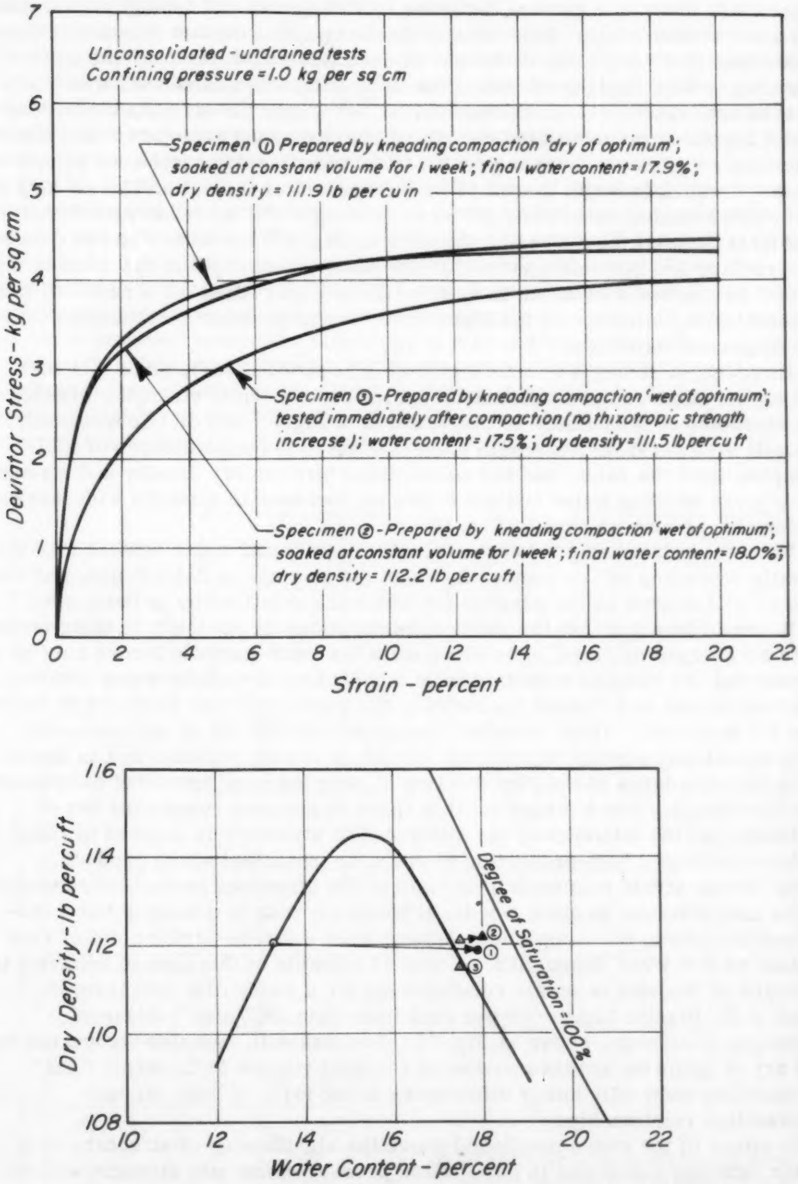

Fig. 11 — INFLUENCE OF MOLDING WATER CONTENT ON STRESS vs STRAIN RELATIONSHIP FOR SAMPLES OF SANDY CLAY.

only a slight effect on the strength values of samples and thus, at a constant value of water content there is not likely to be more than a slight reduction in strength with increase in density for samples compacted wet of optimum.

Furthermore, since all samples in these types of soil would behave as if they had more or less flocculated structures, the stress required to cause 5 per cent strain would be only slightly less than the maximum strength and the relationships between density, water content and strength would be quite similar whether strength were determined at high or low strains.

Typical examples of soils exhibiting this type of relationship between density, water content and strength are shown in Fig. 12(a) for the sandy clay soil and in Fig. 12(b) for a highly plastic clay (Liquid Limit = 59, Plastic Limit = 27, per cent finer than .002 mm. = 46). Behavior of this kind for the sandy clay might possibly have been anticipated since the granular components of the soil will have a major influence on the stress-deformation relationship. However, the relationship between dry density, water content and strength in the as-compacted condition for the highly plastic clay is in marked contrast to that for the silty clay shown in Fig. 10.

It would appear from these data that the relationship between composition and strength of compacted clays cannot be predicted simply on the basis of soil type, and more positive indicator tests are required. At the present time there seems to be no simple criterion for showing which soils will show large effects of a change in structure during compaction and which soils will show little or no effects. It appears that the only way to ascertain the influence of changes in structure on the pore-water pressures and behavior of any given soil is by direct determination in the laboratory.

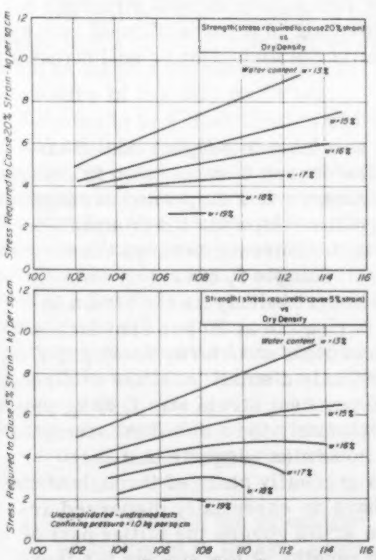

Fig. 12(a)-RELATIONSHIP BETWEEN DRY DENSITY, WATER CONTENT AND STRENGTH AS COMPACTED FOR SAMPLES OF SANDY CLAY-KNEADING COMPACTION.

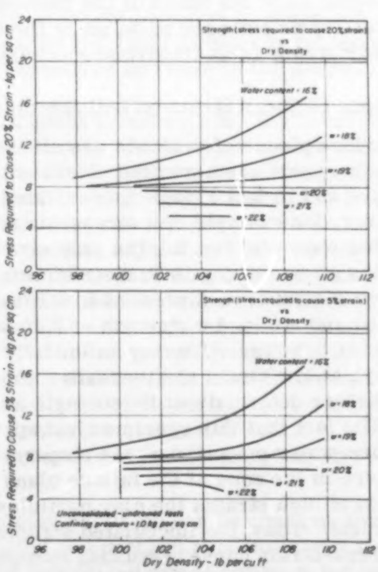

Fig. 12(b)-RELATIONSHIP BETWEEN DRY DENSITY, WATER CONTENT AND STRENGTH AS COMPACTED FOR SAMPLES OF HIGHLY PLASTIC CLAY- KNEADING COMPACTION.

Perhaps one further comment should be made with regard to the undrained strength characteristics of soils in the "as-compacted" condition. There appears to be no fundamental reason why, for samples of the same composition, dispersed and flocculated structures should result in equal strengths at 20 to 25 per cent strain and this may not be true for all soils. In fact, for some soils, samples having dispersed structures may still show lower strengths, even at such high strains, than samples of the same composition having flocculated structures. With such soils there is likely to be a stage at which an increase in density, at any given water content, will change the structure sufficiently to cause a reduction in strength even for strengths determined at high strains. However, the effect will still be much smaller than it would be for 'strengths' measured at low strains.

Effect of Method of Compaction on Soil Properties

So far the data presented have been limited to samples prepared by kneading and impact methods of compaction. It is, in fact, for these methods of compaction that the theory of compaction described on page 1 was developed and for which confirmatory data were obtained. It becomes pertinent to ask at this stage whether the theory would apply to other methods of compaction. Evidence of the different characteristics produced in soils by different methods of compaction. Evidence of the different characteristics produced in soils by different methods of compaction has been available for some time and suggests that different methods of compaction may in fact produce different types of structure. The hypothesis advanced by Lambe does not include a consideration of the effect of the method of compaction on soil structure but can readily be extended to do so by incorporating in it the additional concept of shear strain as an important mechanism in the formation of soil structure.

Influence of Shear Strain on Soil Structure

Various pieces of evidence are already available to suggest that shear strain may produce a marked increase in the degree of dispersion in compacted clays and in fact change a flocculated structure to a dispersed arrangement. Consider, for example, the stress-strain relationships for the compacted samples shown in Fig. 8. The only significant difference between these two samples after soaking is in their structures. Sample 2, having an initially dispersed structure, increased in strength progressively as the strain increased and attained a strength of 2.40 kg per sq cm at 20 per cent strain. Specimen 1, however, having an initially flocculent structure, developed a high strength at low strain and thereafter showed only a small increase in strength with further deformation; its strength at 20 per cent strain was 2.20 kg per sq cm. The fact that this specimen had approximately the same final strength and pore-water pressure as the dispersed specimen suggests that the structure in the zone of the failure plane is gradually changed throughout the test and at high strains the clay particles have an essentially dispersed arrangement. Thus, the flocculated structure would govern the initial part of the stress-strain curve, resulting in high strengths at low strains but thereafter the structure is apparently progressively changed to a dispersed arrangement which controls the strength at high strains.

Again, it has been found that a series of repeated load applications to a compacted clay can produce a marked increase in resistance to deformation,

probably because of a change in structure of the soil.[9] Yet, if the specimen is deformed appreciably after the stiffening effect has been created, the increased resistance to deformation disappears. This might also be interpreted as a destruction of soil structure resulting from shear strain.

Consideration of these effects suggests that if shear strain after compaction can change the structure of a compacted clay, then the shear strains that occur during compaction are also likely to have a profound effect on the initial structure. Furthermore, if large shear strains after compaction can change a flocculated to a dispersed structure, it would seem logical to conclude that large shear strains during compaction can produce a much greater degree of dispersion than would result from a compaction process inducing no appreciable shear strain in the soil.

These concepts are in excellent agreement with the mechanism of soil movement during kneading and impact compaction. When a sample of clay is compacted dry of optimum there is usually no appreciable penetration of the compaction hammer or tamping foot once the soil has been rammed into a compact mass from its original loose condition. In other words, there is no appreciable shear deformation during compaction, with the result that a soil which tends to flocculate will retain a flocculated structure. On the other hand, compaction of the same soil wet of optimum usually results in appreciable penetrations of the compacting hammer or tamping foot, even after the maximum density has been attained, producing considerable shear strain in the soil.

The fact that a marked change in structure can occur in going from slightly dry to slightly wet of optimum suggests that the shear strain during compaction plays an important part in producing an increased dispersion of the clay particles; and that in many cases it is the progressive increase in shear deformation (for a constant compactive effort) as the water content is increased which is largely responsible for the progressive increase in the degree of orientation of the clay particles.

This leads to a modification of the original hypothesis for the development of soil structure during compaction as follows: at low water contents the high electrolyte concentration prevents the double layer from developing fully resulting in low inter-particle repulsion and consequently, for most soils, a tendency for flocculation of the clay particles. An increase in water content causes a decrease in electrolyte concentration, expansion of the double layer, and therefore increased repulsion between clay particles together with a tendency for higher pore-water pressures to develop when the tamping pressure is applied; consequently it leads to a decrease in shear strength. The somewhat reduced tendency to flocculate but more particularly the greater shear deformations under the tamping foot which result from the reduction in shear strength can lead to an increased degree of dispersion or an increased degree of orientation of the clay particles in the compacted soil.

On the basis of this hypothesis it is possible to divide soils into five main classes:

1. Soils in which the orientation of clay particles can be changed simply by an increase in water content during compaction; such soils will tend to have flocculated structures when compacted dry of optimum but will have more dispersed structures when compacted wet of optimum whether the compaction method induces shear strains or not. However, compaction methods inducing shear strain in a soil will cause more dispersion than compaction methods inducing no shear strain.

2. Soils in which the tendency to flocculate is sufficiently great that an increase in water content during compaction will not produce any appreciable increase in dispersion of the clay particles but an increase in water content combined with the influence of large shear strains will produce increased dispersion.
3. Soils in which the tendency to flocculate is so great that neither an increase in water content nor the inducement of shear strains during compaction will produce any appreciable change in structure.
4. Soils which tend to disperse even when compacted dry of optimum; for such soils the structure will remain dispersed when compacted wet of optimum whether the compaction method induces shear strains or not.
5. Soils in which there is a pronounced increase in dispersion when compacted wet of optimum but the influence of structural changes in the clay fraction on the stress-strain characteristics is masked by other factors with the result that the soil behaves from a stress-strain point of view almost as if no change in structure had occurred.

The above hypothesis leads to an understanding of the effect of method of compaction on soil properties. Furthermore, the validity of the hypothesis with regard to the effect of shear deformation can readily be determined. If two samples of a soil in Class 1 or 2 above could be compacted at the same water content to the same dry density, first using a procedure involving large shear deformations and then using a procedure involving negligible shear deformations, it would be expected that the first sample would have a relatively dispersed structure while the second specimen would have a relatively flocculated or less dispersed structure.

A procedure involving large shear deformations is the use of kneading compaction for samples wet of optimum. A procedure involving no appreciable shear strain is the compaction of a soil in a mold using a ram which covers the entire area of the mold, a process which has been termed static compaction. Hence, a comparison of the properties of samples prepared by kneading and static compaction should throw some light on the basic cause of soil structure and on the influence of compaction methods on soil properties. For samples compacted dry of optimum both static and kneading compaction should produce no appreciable shear deformation in the soil and consequently result in similar structures (relatively flocculated or with random orientations of clay particles). However, for samples compacted wet of optimum kneading compaction should produce appreciable shear deformations resulting in a more dispersed structure or a greater degree of particle orientation, while static compaction and the associated negligible shear deformation should still produce a relatively flocculated structure. For samples of the same dry density and water content the two methods of compaction should therefore result in similar properties for samples prepared dry of optimum but in markedly different properties for samples compacted wet of optimum.

Comparison of the Effects of Kneading and Static Compaction on Swell and Shrinkage

In order to investigate the above conclusions, tests were conducted to determine the swell and shrinkage characteristics of samples of sandy clay and silty clay prepared by kneading and static compaction.

Fig. 13 shows a comparison of the amounts of swell and shrinkage for samples of sandy clay compacted dry and wet of optimum. It will be seen that

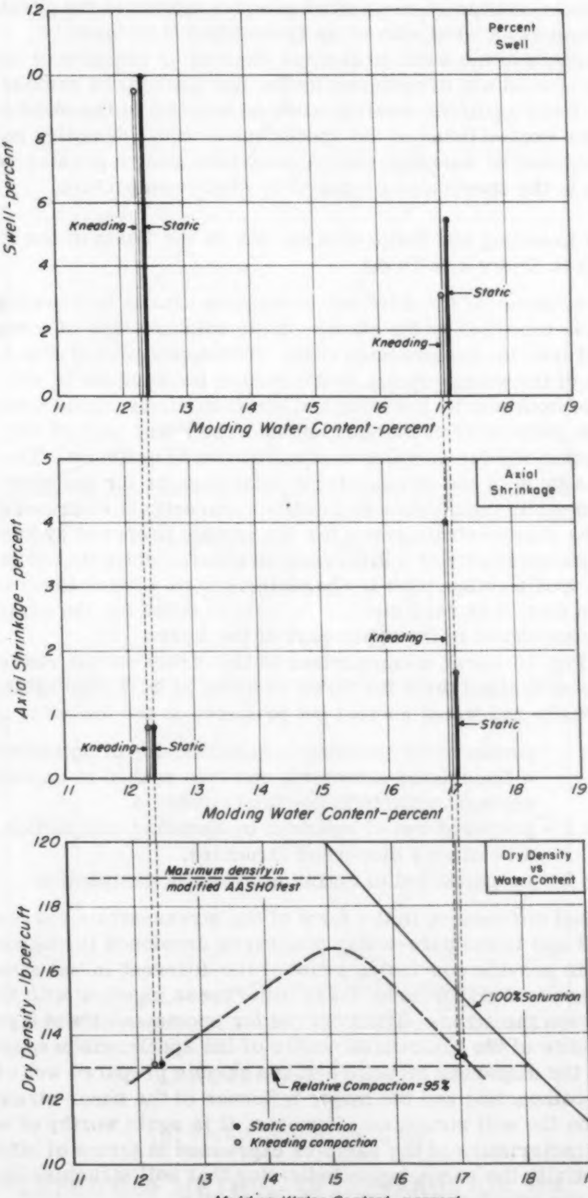

Fig.13-SWELL AND SHRINKAGE FOR SAMPLES OF SANDY CLAY PREPARED BY KNEADING AND STATIC COMPACTION.

samples compacted dry of optimum by the two methods of compaction have similar swelling and shrinkage characteristics; but for samples compacted wet of optimum, statically compacted samples exhibited the greater swell and less shrinkage associated with more flocculated structures.

A comparison of the swell pressures exerted by samples of sandy clay and silty clay prepared wet of optimum by the two methods of compaction is shown in Fig. 14. Here again the swell pressures exerted by the statically compacted specimens exceed those of the specimens of equal densities and water contents prepared by kneading compaction, indicating a greater degree of flocculation in the specimens prepared by static compaction.

Influence of Kneading and Static Compaction on the Form of the Stress-Strain Relationship in Undrained Tests

Further evidence of the different structures caused by kneading and static compaction is provided by the stress-strain relationships of compacted clays in undrained triaxial compression tests. The upper part of Fig. 15 shows a comparison of the stress-strain relationships for samples of silty clay compacted dry of optimum by kneading and static methods, again indicating the similarity in properties of the specimens. The lower part of Fig. 15 shows a similar comparison for samples compacted wet of optimum. The marked difference in form of the stress-strain relationships for samples prepared by kneading and static compaction is readily apparent. Furthermore, it will be noted that the stress-strain curve for the sample prepared by kneading compaction is characteristic of a dispersed structure, while that of the sample prepared by static compaction is characteristic of a more flocculated structure; in fact, it is very similar in form to those for the samples prepared dry of optimum shown in the upper part of the figure.

Finally, Fig. 16 shows a comparison of the stress-strain curves in consolidated-undrained tests for three samples of silty clay having essentially the same density and water content but prepared in the following manner:

Specimen 1 - prepared by kneading compaction dry of optimum, producing a flocculated structure, and then soaked at approximately constant volume to the final condition.

Specimen 2 - prepared wet of optimum by kneading compaction, producing a relatively dispersed structure.

Specimen 3 - prepared wet of optimum by static compaction.

The marked differences in the form of the stress-strain curves for specimens 2 and 3 and in the pore-water pressures developed throughout the tests would seem to provide convincing proof of the different initial structures of these specimens. Furthermore, these differences together with the similarity in form between the stress-strain curves for specimens 1 and 3 provide further evidence of the flocculated nature of the specimens prepared by static compaction, the dispersed structure of the sample prepared wet of optimum by kneading compaction and the major influence of the shear strain during compaction on the soil structure. However, it is again worthy of note that the strength characteristics of the samples expressed in terms of effective stresses are essentially the same, again indicating that soil structure has no influence on the effective strength characteristics.

It should also be noted that the different methods of compaction will not produce such a marked difference in stress-deformation characteristics in

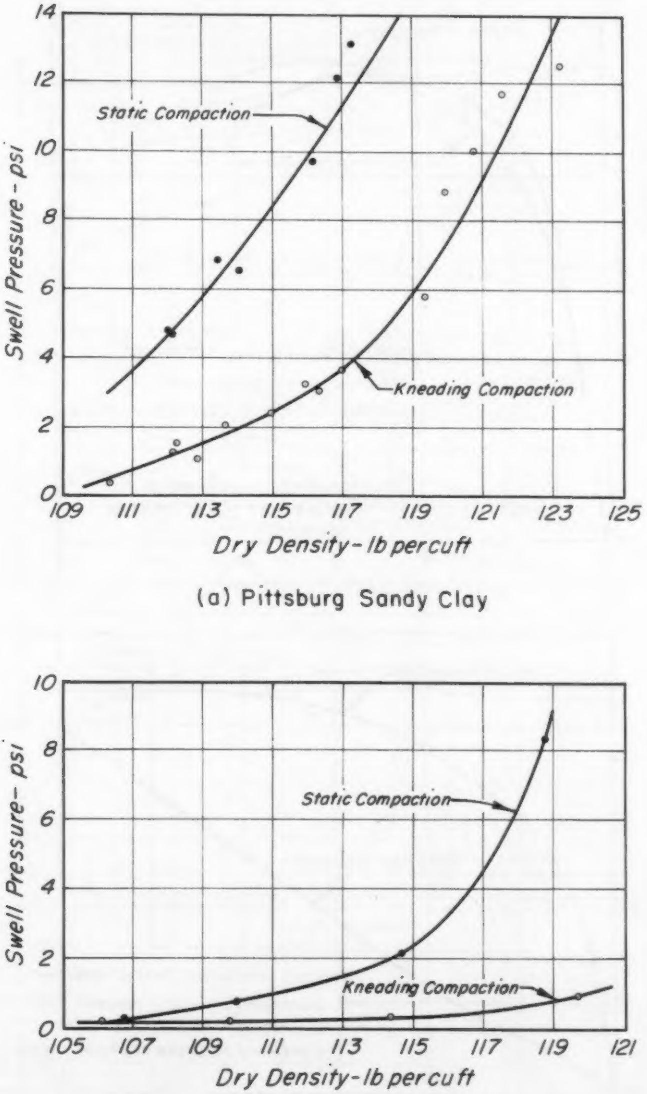

Fig.14 - EFFECT OF METHOD OF COMPACTION ON SWELL PRESSURE FOR SAMPLES COMPACTED TO HIGH DEGREE OF SATURATION.

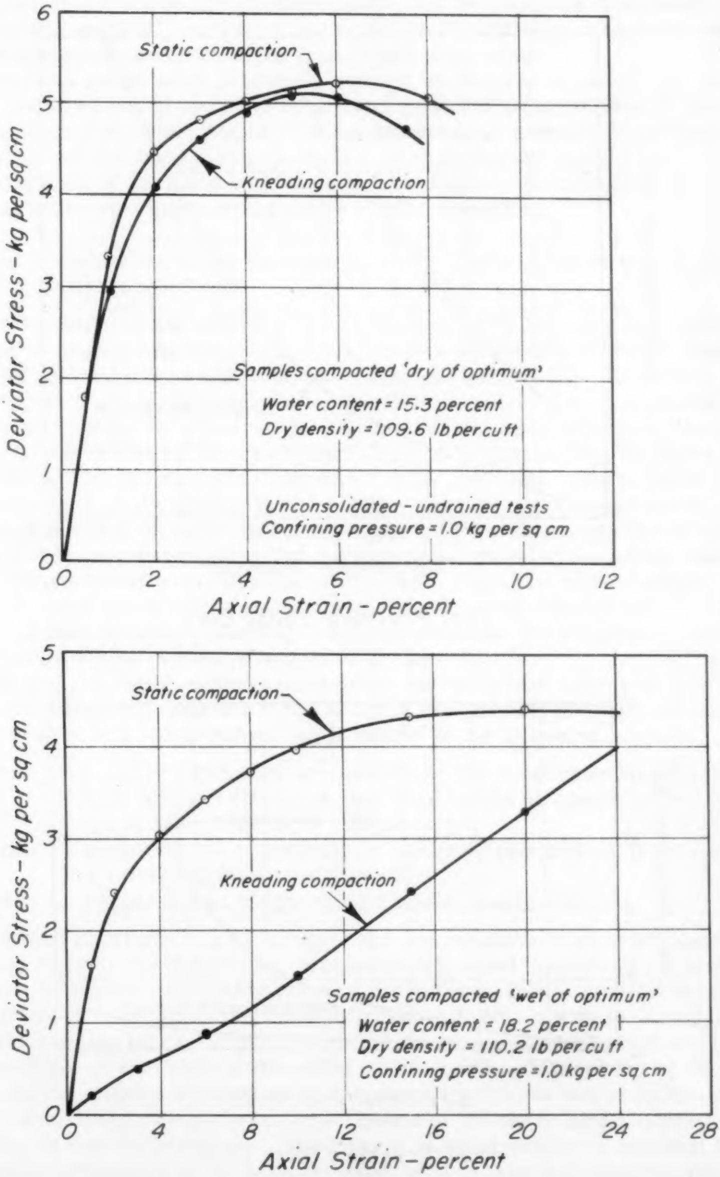

Fig.15 - STRESS vs DEFORMATION RELATIONSHIPS FOR SAMPLES OF SILTY CLAY PREPARED DRY AND WET OF OPTIMUM BY KNEADING AND STATIC COMPACTION.

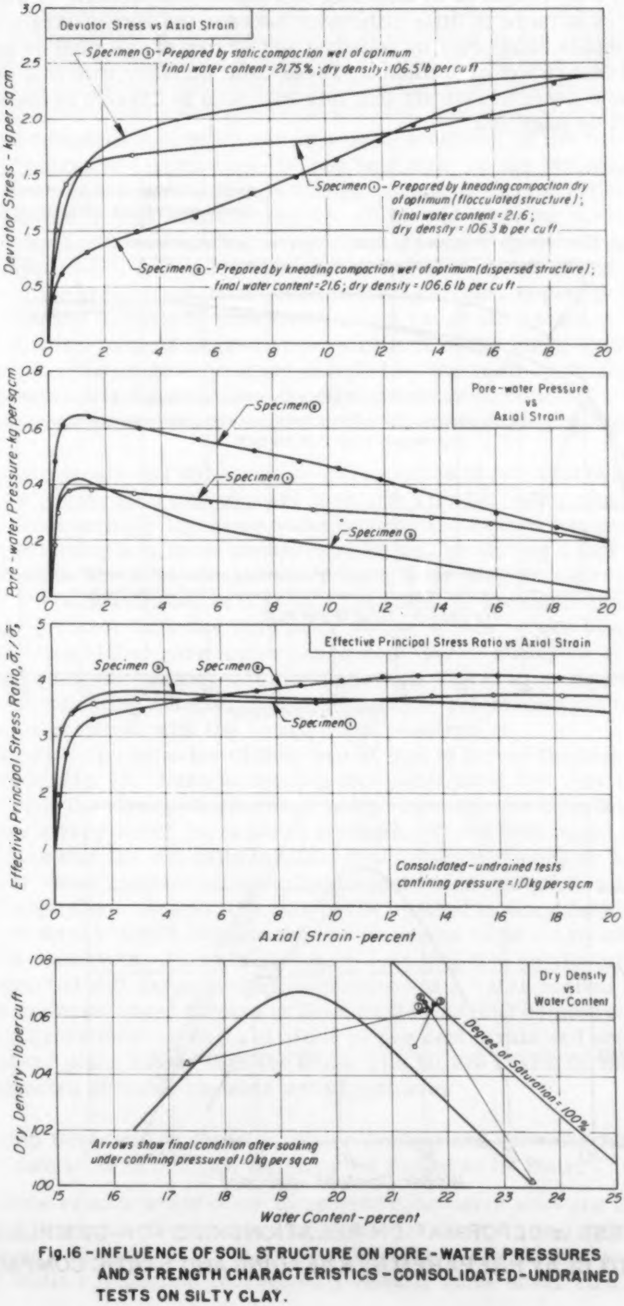

Fig.16 - INFLUENCE OF SOIL STRUCTURE ON PORE - WATER PRESSURES AND STRENGTH CHARACTERISTICS - CONSOLIDATED-UNDRAINED TESTS ON SILTY CLAY.

all types of soil. Fig. 17 shows the stress-strain relationships for samples of the sandy clay prepared by kneading and static compaction. It will be noted that in this case there is little difference between the stress-strain relationships of samples compacted by kneading wet or dry of optimum or compacted by static pressure wet of optimum. These data, together with that shown in Fig. 13, would seem to indicate that this soil falls in Class 5 of the suggested subdivisions on page 108.

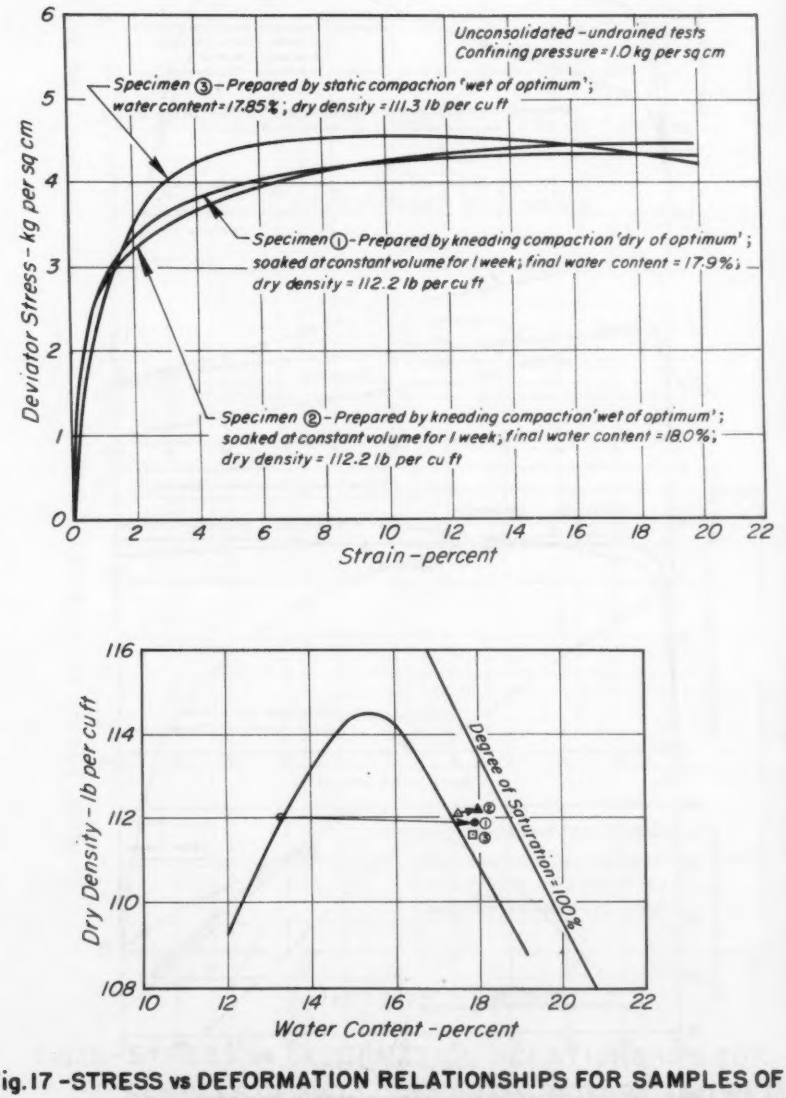

Fig. 17 - STRESS vs DEFORMATION RELATIONSHIPS FOR SAMPLES OF SANDY CLAY PREPARED BY KNEADING AND STATIC COMPACTION.

Influence of Foot Penetration During Kneading Compaction

Another method of investigating the influence of shear deformation on the structure and behavior of compacted soils is provided by the variations in penetration of the tamping foot during soil compaction by kneading methods. If a sample is compacted wet of optimum then there is a theoretical limit for the maximum dry density which may be attained; this limit corresponds to a condition of complete saturation and is usually indicated by the "zero air voids curve" or "degree of saturation = 100 per cent line" on the dry density vs. water content plot. In actual fact, a sample cannot be compacted to this condition because it is impossible to remove all the air from the voids by compaction methods and for kneading compaction the upper limit will usually be that corresponding to a degree of saturation between 90 and 95 per cent.

This limiting condition will be reached for a certain tamping pressure but further increases in tamping pressure will not cause any increase in density; they will, however, result in greater penetrations of the tamping foot into the soil. Thus, it is possible to prepare samples by the same method of compaction (kneading) to the same dry density and water content but with different amounts of shear strain being induced in the samples during the compaction process.

The stress-strain and shrinkage characteristics of two series of samples of silty clay prepared in this way are shown in Fig. 18. After compaction the samples had essentially the same water content and density but one series was prepared using a tamping pressure of 80 psi. producing a foot penetration of 0.05 in. while the other was prepared using a tamping pressure of 320 psi. producing a foot penetration of 0.43 in. The samples for which the foot penetration was greatest exhibited the flatter stress-strain curves in undrained tests (indicating higher pore-water pressures), lower strengths at 5 per cent strain and higher shrinkages—all manifestations of a greater degree of dispersion and indicating a higher degree of particle orientation in these samples than in those prepared with the smaller foot penetration.

In contrast to the behavior of this type of soil is that of the highly plastic clay shown in Fig. 19. Here increasing foot penetration had very little effect on the form of the stress strain relationship, although the largest penetration did produce a very small decrease in strength. These data might be interpreted to indicate that the soil falls in Class 3 of the subdivisions on page 108 and that even at water contents above optimum and under compaction conditions inducing large shear strains, the clay particles tend to flocculate, resulting in the form of stress-strain relationship which seems to be characteristic of flocculated conditions. It is interesting to note that this particular soil, even when compacted with large shear deformations at a water content 16 per cent above the optimum water content in the modified AASHO compaction test, still develops its maximum strength at about 10 per cent strain and shows a well-defined shear plane rather than the bulge type failure characteristics of many soils compacted at water contents wet of optimum.

Relationship Between Dry Density, Water Content and Undrained Strength in the "As-Compacted" Condition for Samples Prepared by Static Compaction

The above results would seem to provide conclusive evidence that the shear strain during compaction can have an important effect on soil structure and therefore on soil properties. It follows from these results that samples prepared by static compaction undergo only slightly more shear strain when they

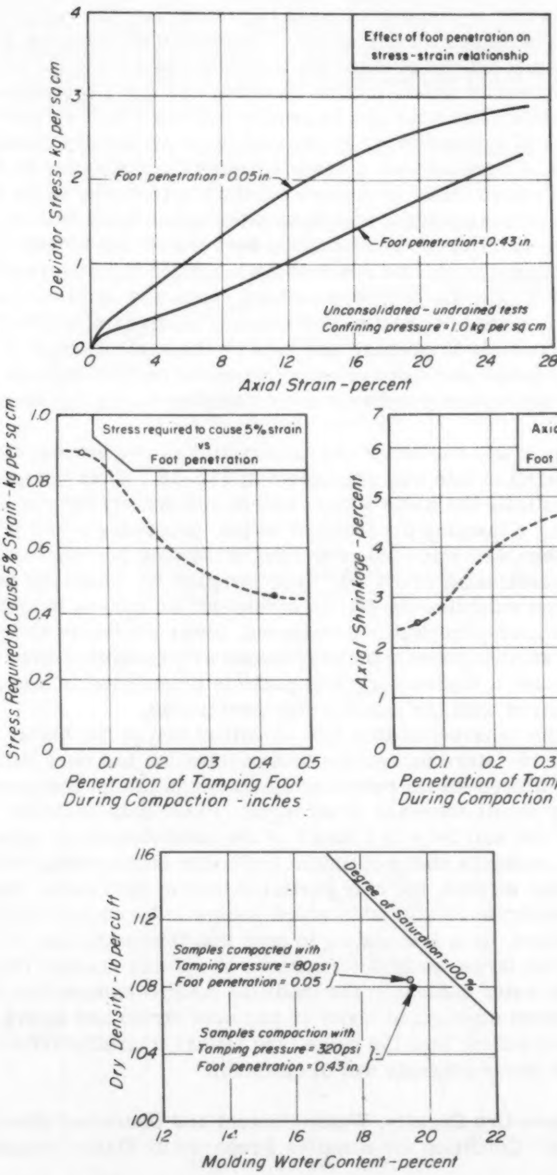

Fig. 18 – EFFECT OF STRAIN DURING COMPACTION ON DEFORMATION AND SHRINKAGE CHARACTERISTICS — SILTY CLAY.

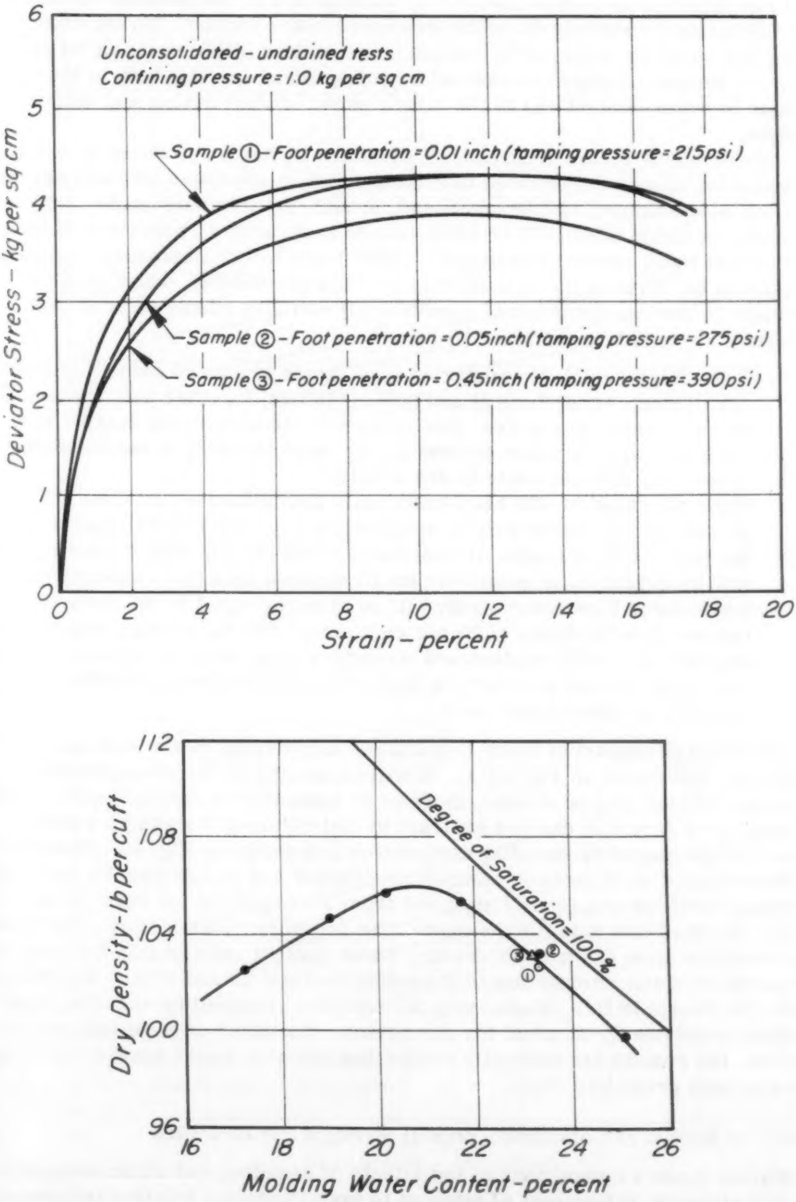

Fig.19—EFFECT OF STRAIN DURING COMPACTION ON STRESS vs STRAIN RELATIONSHIP—HIGHLY PLASTIC CLAY.

are compacted wet of optimum than when they are compacted dry of optimum and any differences in structure are primarily due to the reduced tendency for flocculation resulting from the increased water content. On the other hand, for samples prepared by kneading compaction, differences in structure between samples compacted wet and dry of optimum are due both to the increase in water content and to the larger shear strains during wet side compaction.

Since a major factor in dispersed structure formation is missing in static compaction, it may be expected that differences in structure between dry side and wet side samples will be small and, in fact, that for most soils compacted by static methods there will be little change in structure regardless of the compacted water content and density. This leads to two important conclusions regarding the form of the relationship between dry density, water content and strength in the "as-compacted" condition for samples prepared by static compaction:

1. Since the strength of compacted clay seems to depend primarily on the dry density, water content and soil structure, together with the associated pore-water pressures, (and changes in structure are small), at a constant value of water content the strength is likely to increase progressively with increase in dry density.
2. Since all samples will have essentially flocculated structures, the stress-strain curves will be relatively steep in the initial stages and the maximum strengths or resistances close to the maximum strength will be developed at relatively small strains; thus, the 'strength' determined at 5 per cent strain will be close or equal to the strength determined on the basis of 20 per cent strain and the relationship between dry density, water content and strength will be similar in form regardless of the strain at which the strength is determined (provided it is larger than about 5 per cent).

Evidence in support of these conclusions is provided by the test data for the silty clay soil shown in Fig. 21(a). It will be noted that for strengths determined at high or low strains, there is no reduction in strength with increase in density—a result in marked contrast to that obtained for samples of the same soil prepared by kneading compaction and shown in Fig. 10. The effects of the different structures in samples compacted wet of optimum by static and kneading methods are readily apparent from a comparison of Figs. 10 and 21(a). Further test data for the sandy clay and highly plastic clay soils showing the same form of strength-density-water content relationship for samples prepared by static compaction is presented in Figs. 20 and 21(b). For these soils the change in this relationship for samples prepared by kneading compaction is not nearly so great for the reasons discussed on page 103. Nevertheless, the results for statically compacted samples would seem to confirm the concepts presented above.

Effect of Method of Compaction on Soil Strength and Structure

Having made a comparison of the effects of kneading and static compaction on soil strength, it becomes of interest to investigate the relative influences of other methods of compaction. The most widely used laboratory compaction procedure is undoubtedly that involving the dropping of a hammer from a given height onto the surface of the loose soil, a process which has been termed

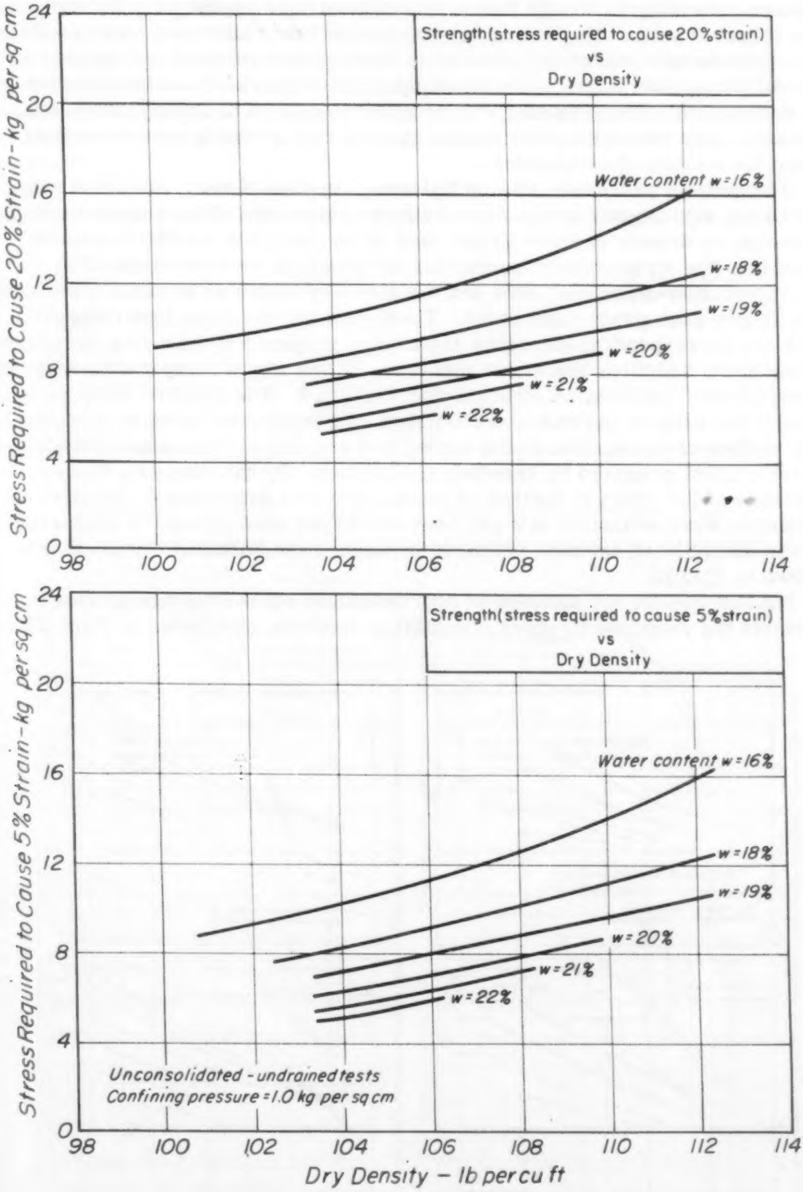

Fig. 20-RELATIONSHIP BETWEEN DRY DENSITY, WATER CONTENT AND STRENGTH AS COMPACTED FOR SAMPLES OF HIGHLY PLASTIC CLAY — STATIC COMPACTION.

impact compaction. In recent years vibratory methods of compaction have also been developed, though they have not been used appreciably for compacting cohesive soils. To provide information on the relative influence of different methods of compaction, a series of tests were conducted on samples of the silty clay soil prepared by kneading, impact, vibratory and static methods of compaction. The silty clay soil was selected for this study since it is apparently very susceptible to changes in structure and properties resulting from the method of compaction.

The general procedure was as follows. Samples of silty clay were prepared by, say, impact compaction, using a compactive effort which would give a maximum density of about 95 per cent of the modified AASHO maximum density. The stress-strain curves for the samples were determined by unconsolidated-undrained tests and the stresses required to cause 5 per cent and 20 per cent strain were noted. These "strengths" were then compared with the corresponding strengths of samples prepared by kneading compaction to the same densities and water contents. In this way a comparative strength index termed the relative strength was evaluated. The relative strength is simply the ratio of the undrained strength of a compacted sample prepared by any method of compaction to the strength of a sample of the same density and water content prepared by kneading compaction. By this means a simple measure of the effect of method of compaction was determined. Relative strengths were evaluated at 5 per cent and 20 per cent strain. A comparison of the strengths of samples prepared by impact and kneading compaction is shown in Fig. 22.

Similar results for samples of approximately equal densities and water contents but prepared by other compaction methods, are shown in Figs. 23

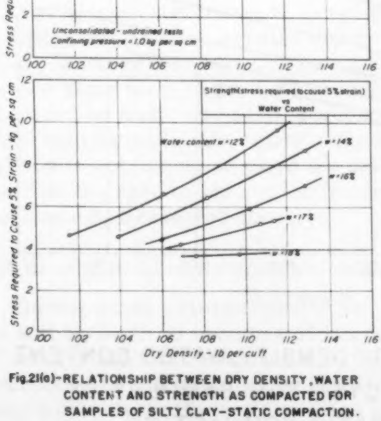

Fig.21(a)-RELATIONSHIP BETWEEN DRY DENSITY, WATER CONTENT AND STRENGTH AS COMPACTED FOR SAMPLES OF SILTY CLAY-STATIC COMPACTION.

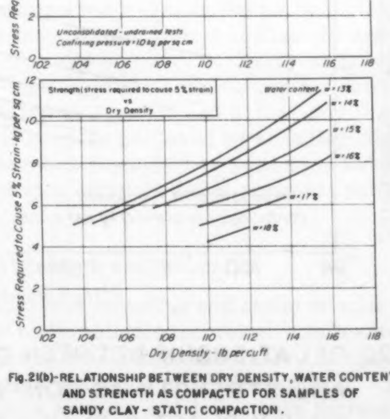

Fig.21(b)-RELATIONSHIP BETWEEN DRY DENSITY, WATER CONTENT AND STRENGTH AS COMPACTED FOR SAMPLES OF SANDY CLAY - STATIC COMPACTION.

and Fig. 24. Fig. 23 shows a comparison of the effects of vibratory and kneading compaction; Fig. 24 shows a comparison of the effects of static and kneading compaction. It will be noted that dry of optimum, the relative strength values are always close to unity, indicating that the method of compaction has little effect on the soil strength. Wet of optimum, the relative strength values determined at 5 per cent strain vary widely for different methods of compaction but only small differences are apparent for strengths determined at high strains.

Since the strength comparisons shown in Figs. 22, 23 and 24 all involve kneading compaction as a basis for comparison, and since the compaction

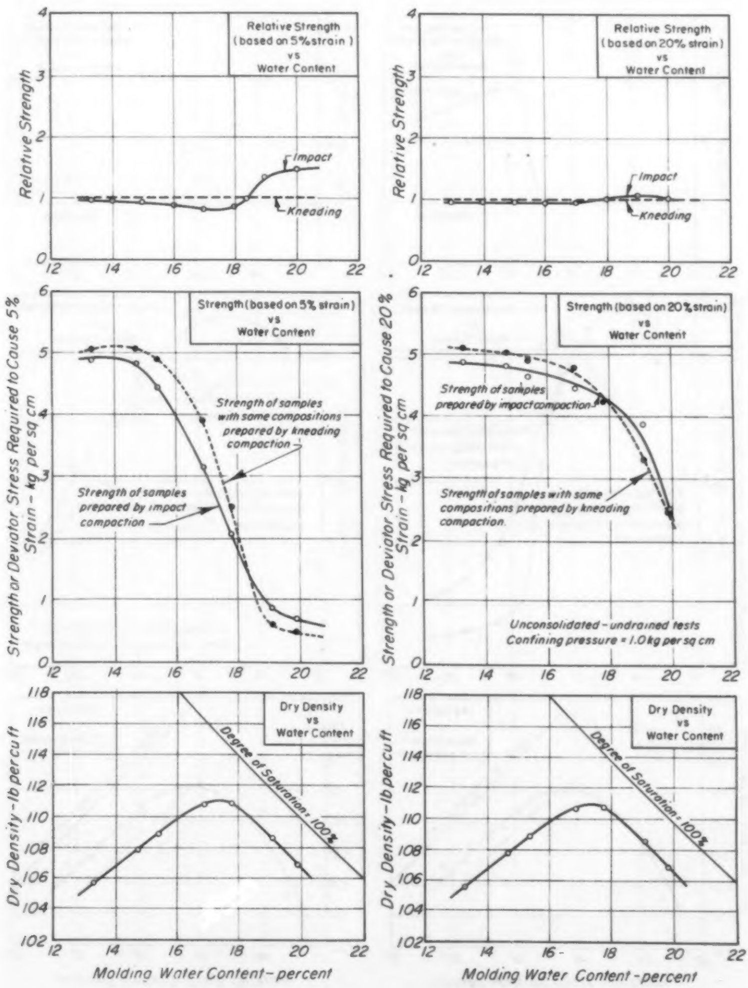

Fig. 22 - COMPARISON OF STRENGTHS OF SILTY CLAY SAMPLES PREPARED BY KNEADING AND IMPACT COMPACTION.

curves for which the comparisons are made are closely alike, a comparison of the effects of the four different methods of compaction can be obtained by replotting the relative strength curves on one diagram. This is shown in Fig. 25(a). This type of plot emphasizes the fact that for this soil at least, the method of compaction has little effect on the strength of samples compacted dry of optimum. For samples compacted wet of optimum, the influence of the method of compaction is considerable and the strength of samples of the same composition increases in the following order of compaction methods: kneading, impact, vibratory, static. This would seem to indicate that the degree of orientation of the clay particles and the pore-water pressures

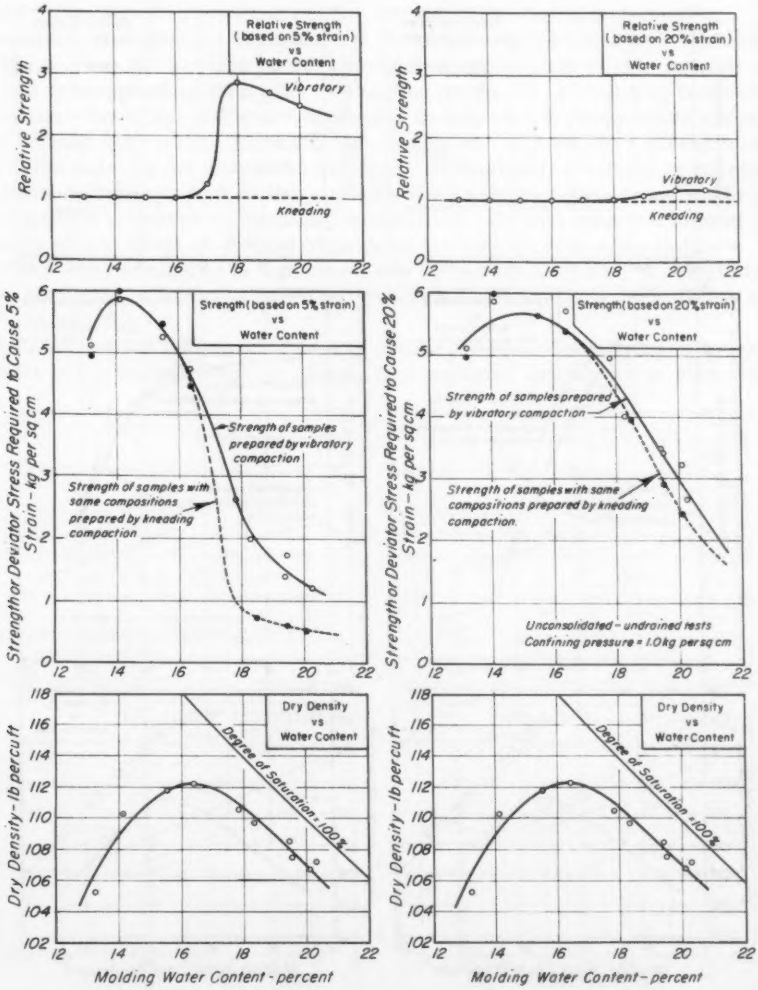

Fig.23-COMPARISON OF STRENGTHS OF SILTY CLAY SAMPLES PREPARED BY KNEADING AND VIBRATORY COMPACTION.

decrease in the same order so that the more flocculated structures give the highest strengths.

Further evidence of this is provided by the shrinkages of samples prepared by the different methods of compaction. A comparison of the effect of method of compaction on the 'strengths' determined at low strains and on the shrinkage of the silty clay is shown in Fig. 26. It will be seen that both strengths and shrinkage are essentially the same for samples compacted dry of optimum. Wet of optimum both the strength and shrinkage data are consistent with the concept of increasing particle orientation in the order of compaction methods: static, vibratory, kneading.

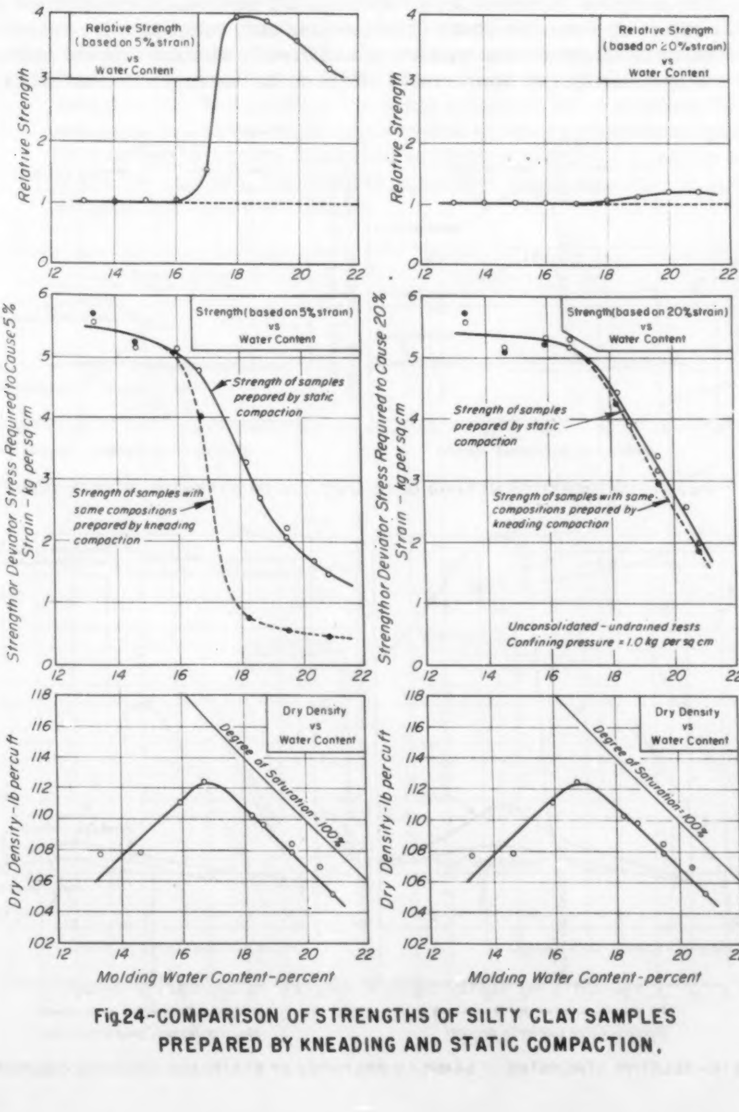

Fig.24-COMPARISON OF STRENGTHS OF SILTY CLAY SAMPLES PREPARED BY KNEADING AND STATIC COMPACTION.

Finally, it will be noted from Fig. 25(a) that even for samples compacted wet of optimum, the strengths determined at high strains are not appreciably affected by the method of compaction, indicating that the initial differences in structure become less significant as the strain increases, probably due to the fact that the shear strain in the zone of failure of the sample is changing the original structure to a more dispersed arrangement and all the samples, whatever their original structure, are developing dispersed orientations at large strains.

In the light of the above discussion it would appear that the data presented in Fig. 25(a) and 26 might reasonably be interpreted as follows:

a. For samples prepared dry of optimum all methods of compaction produce no appreciable shear deformations and, consequently, essentially flocculated structures which are sufficiently similar that the method of compaction has no appreciable effect on the strength characteristics.

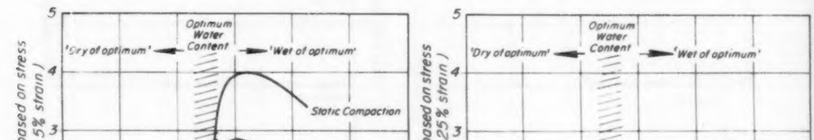

Fig.25(a)-INFLUENCE OF METHOD OF COMPACTION ON STRENGTH OF SILTY CLAY.

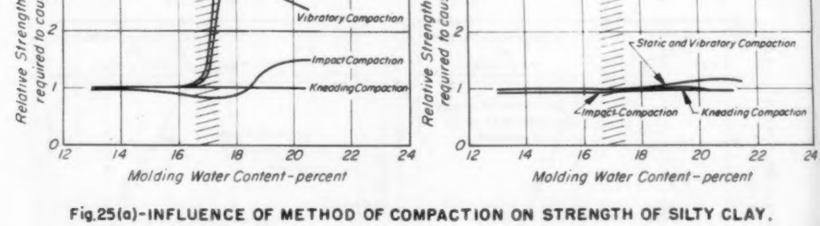

Fig. 25 (b)-RELATIVE STRENGTHS OF SAMPLES PREPARED BY STATIC AND KNEADING COMPACTION

b. For samples of the same composition prepared wet of optimum:

1. Kneading compaction causes the largest shear strains during compaction and, therefore, the highest degree of dispersion, the highest pore-water pressures and the lowest strengths at low strains, and the highest shrinkage.
2. Impact compaction causes slightly less shear strain during compaction and consequently the degree of dispersion is not quite so great as for kneading compaction, the strengths at low strains are slightly higher and the shrinkage is slightly less.
3. Static compaction causes little shear strain during compaction, resulting in a relatively flocculated structure, the lowest pore-water pressures and highest strength at low strains, and least shrinkage.
4. Vibratory compaction over the entire area of a sample, as used in these tests, should give little chance for shear strain to occur in the samples and thus produce the same structure as is obtained by static compaction. However, it appears that the vibrations enable particles to reorient to a more dispersed arrangement than is possible with static compaction, resulting in somewhat lower strengths at low strains and higher shrinkage.

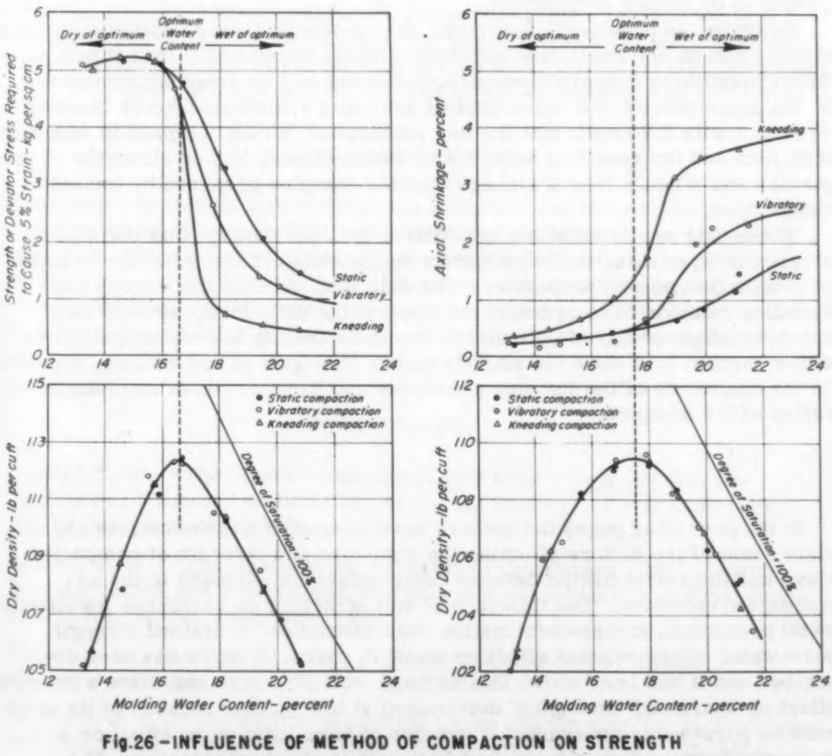

Fig.26 – INFLUENCE OF METHOD OF COMPACTION ON STRENGTH AND SHRINKAGE OF SILTY CLAY.

Since the greatest differences in the effects of the various compaction methods occur between the undrained strengths of samples prepared by kneading and static methods, it is interesting to evaluate the magnitude of this difference for a variety of soils. Fig. 25(b) shows the relative strengths of samples prepared by static and kneading compaction for three soils. It will be seen that the effect of method of compaction can vary widely with different soils; in some cases it may cause a strength variation as great as 400 per cent, but in others the effect may be quite small. It is interesting to note, however, that even for strengths determined at high strains, for each of these soils the strengths of some samples prepared by static compaction are as much as 20 per cent greater than those of the same composition prepared by kneading compaction.

Finally, it is worthy of note that the differences in structure and strength resulting from impact and kneading methods are apparently small. A check on the shear strains during compaction for samples of the silty clay at a water content of 20.4 per cent (3 per cent wet of optimum) showed that the falling hammer during impact compaction penetrated about 0.12 inches into the sample while the tamping foot during kneading compaction penetrated 0.18 inches—the greater shear strain during kneading compaction leading to samples having lower strengths at low strains and higher shrinkage than those prepared by impact compaction.

However, as previously described, for compaction wet of optimum the same density can be obtained using different tamping pressures. Thus it was also found possible to prepare other samples of the soil by kneading compaction to the same density and water content but using a somewhat lower tamping pressure with the result that the foot penetration during compaction was only 0.04 inch and the resulting samples exhibited slightly higher strengths at low strains and slightly less shrinkage than the samples prepared by impact compaction.

These data are in excellent accordance with the concept that the shear strain during compaction is primarily responsible for the influence of method of compaction on soil properties. The data also indicate that impact and kneading methods of compaction are apparently sufficiently similar that samples prepared wet of optimum to the same density and water content by either method may show the slightly higher strengths at low strains, depending on the magnitude of the tamping pressures and hammer blows used for preparation of the samples.

CONCLUSIONS

In the preceding pages the authors have attempted to demonstrate and explain some of the factors affecting the structure and strength of compacted clays and the relationships between composition and strength in the as-compacted condition. The influence of soil structure on shrinkage, swelling, swell pressures, stress-deformation characteristics, undrained strength, pore-water pressures and effective strength characteristics has been described and it has been shown that although soil structure may have a profound effect on undrained 'strengths' determined at low strains because of its influence on pore-water pressures, it appears to have little or no effect on soil strength characteristics expressed in terms of effective stresses. The influence of the strain at which undrained soil strength is determined on the

relationship between composition and strength has been demonstrated, and typical examples of this relationship, illustrating the influence of changes in soil structure on the form of the results obtained, have been presented.

It has also been shown that the structure developed in a compacted soil is greatly influenced by the shear strains induced in the soil during the compaction process; such strains apparently tend to produce a dispersed arrangement of soil particles and thus, for soils in which the interparticle forces are not so great that flocculation will occur under all compaction conditions, methods of compaction inducing shear strains produce a greater degree of particle orientation, lower strengths at low strains in undrained tests, greater shrinkage and less swelling than methods of compaction inducing little shear strain. As a consequence of this effect, different methods of compaction tend to produce similar characteristics in samples compacted dry of optimum to any given density and water content but produce different characteristics in samples compacted wet of optimum. For samples compacted wet of optimum to any given density and water content, particle orientation and shrinkage tend to decrease and strength at low strains tends to increase in the following order of compaction methods: kneading, impact, vibratory, static.

The paper represents the first of a series of two dealing with the strength of compacted clays; a more detailed consideration of the principles described above to the strength characteristics of compacted clays after soaking and the use of stress vs strain relationships for predicting the influence of molding water content, method of compaction and strain at failure on the undrained strength of compacted clays will be covered in the second paper of the series.

ACKNOWLEDGEMENT

The authors gratefully acknowledge the assistance of their colleagues and staff of the Soil Mechanics and Bituminous Materials Laboratory, University of California, in the preparation of this paper: Professor J. K. Mitchell prepared and analyzed the thin sections of kaolinite and, with Professor C. L. Monismith, critically reviewed the manuscript; Messrs. A. Buchignani and L. Shifley conducted many of the tests; and Mr. G. Dierking prepared the figures.

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Journal of the

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

DESIGN OF UNDERSEEPAGE CONTROL MEASURES FOR DAMS AND LEVEES

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This is one of three companion papers on underseepage and is referred to as companion paper No. 2. The first paper is entitled "Investigation of Underseepage—Mississippi River Levees" and is referred to as companion paper No. 1. The third paper is entitled "Construction and Maintenance of Underseepage Control Measures" and is referred to as companion paper No. 3.

SYNOPSIS

Methods of controlling seepage and excessive hydrostatic pressures beneath dams and levees founded on deep strata of pervious sands are presented in this paper. The principles involved in the different methods of underseepage control are also considered. Although the design of seepage control measures is not an exact science, formulas and criteria for preparing a rational design of such are given. These are based on known seepage laws, laboratory tests, and field observations.

INTRODUCTION

The control of underseepage and prevention of sand boils landward of levees founded on deep strata of pervious sands require some measure that will control erosional seepage and reduce excess pressure beneath the landside top stratum to a safe value.

Note: Discussion open until March 1, 1960. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2217 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 5, October, 1959.

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Methods that may be used to control seepage are impervious riverside blankets, relief wells, landside berms, drainage trenches, cutoffs, and sublevees. The choice of a control measure depends upon a number of factors, including the character of the foundation, cost, permanency, availability of right of way, maintenance, and disposal of seepage water. The principles involved in each of these methods of control are quite different. Where the pervious substratum is exposed riverward of a levee, an impervious riverside blanket acts to control seepage by increasing the resistance to seepage entry into the pervious substratum, thereby decreasing both seepage flow and excess pressure landward of the levee. An impervious cutoff beneath a levee blocks the passage of seepage beneath the levee even though there is a ready entry for seepage into the pervious foundation through the river channel or riverside borrow pits. Instead of blocking the flow of seepage beneath a levee, relief wells along the landside toe of a levee provide pressure relief and controlled seepage outlets that offer little resistance to flow but at the same time prevent erosion of the soil. A landside berm controls underseepage by increasing the thickness of the top stratum immediately landward of the levee so that the combined weight of the berm and top stratum is adequate to resist the excess uplift pressure, and by increasing the path of seepage flow through the pervious aquifer to the extent that the residual excess pressure at the toe of the berm is no longer critical. Filling sublevee basins with water reduces the activity and danger of sand boils by counterbalancing the excess head beneath the top stratum in the area encompassed by the sublevee. A drainage trench controls seepage by intercepting it as it emerges from the pervious substratum without allowing erosion to take place. It also provides a certain amount of pressure reduction landward of a levee where the blanket or trench contacts the underlying aquifer.

For reasons subsequently discussed, only riverside blankets, relief wells, and seepage berms are generally recommended for the control of seepage beneath levees along the middle and lower reaches of the Mississippi River.

Seepage control measures are considered necessary where observed or estimated values of h_o* may be expected to equal or exceed h_c (approximately 0.75 z_t) at design flood stages. If seepage control measures are considered necessary, they should be designed in accordance with the following criteria:

a. For levees with a semipervious top stratum landward of levee:

(1) Riverside blankets.—Where no control measures are present, riverside blankets should be designed so that i at the toe of the levee does not exceed 0.5 to 0.6. Where landside berms wider than 150 ft are present, but additional control measures are considered necessary, riverside blankets should be designed so that i at the toe of the berm does not exceed 0.6 to 0.7.

(2) Relief Wells.—Where no control measures are present, relief wells should be designed so that i_{max} midway between wells or landward from the well line does not exceed 0.5 to 0.6. Where landside berms wider than 100 ft are present, but additional control measures are considered necessary, relief wells should be designed so that i_{max} = 0.6 to 0.7.

(3) Seepage Berms.—Seepage berms should have a width and thickness such that i through the top stratum and berm at the landside toe of the levee will not exceed 0.5, and i at the berm toe will not exceed

*See Appendix A for definition of notations and symbols.

0.75 to 0.80. However, seepage berms need not have a width exceeding 300 to 400 ft depending on soil conditions and height of levee.

b. For levees with no natural top stratum landward of levees:

(1) Riverside Blankets.—If creep ratio is less than values given by Bligh and Lane and Q_s at project flood stage would be excessive (say greater than about 200 gpm per 100 ft of levee), riverside blankets should be designed to reduce Q_s to an acceptable amount. The following are the minimum values ordinarily given for Bligh's and Lane's creep ratios, respectively: Very fine sand or silt - 18, 8.5; fine sand - 15, 7; medium sand - --, 6; coarse sand - 12, 5; fine gravel or sand and gravel - 9, 4; coarse gravel including cobbles - 4 to 6, 3; boulders with some cobbles and gravel - --, 2.5.

(2) Relief Wells.—If creep ratio is too low and natural seepage Q_s is greater than about 200 gpm per 100 ft of levee, relief wells should be designed to intercept enough seepage so that the uncontrolled seepage emerging landward of the levee will not be more than about 150 to 200 gpm.

(3) Seepage Berms.—If creep ratio is less than values given by Bligh and Lane, length of berm should be such as to increase the creep ratio to an acceptable value, and i through the berm at toe of levee to a value equal to or less than 0.5. The values by Bligh and Lane for various soils are shown in paragraph b(1) above.

Riverside Blankets

An impervious riverside blanket can be used to reduce the intensity of seepage and pressures landward of a levee where the pervious substratum is, or is nearly, exposed riverward of the levee. Such blankets are particularly adapted to situations where no top stratum exists riverward of the levee or where most of the natural top blanket has been removed in borrow operations.

Correct design of a riverside blanket requires determination of the extent, type, thickness, and permeability of the existing blanket. The first three of these items can be determined from surveys and borings; the permeability for the various types of soil materials can then be estimated on the basis of existing field data.

Where the blanket is to be developed by means of abatis dikes, the new fill will probably consist of silts or silty sands, depending upon velocity and flow conditions along the levee during high water. On the basis of experience and actual field measurements, the permeability of such a blanket would probably be about 1 to 2 x 10^{-4} cm per sec.

Where haul-in construction is contemplated, reasonable estimates of the permeability of blanket materials can be obtained from laboratory tests on compacted samples.

Formulas for the design of riverside blankets for various conditions are presented in the following paragraphs.

Case I. **No Natural Riverside Top Stratum**

a. **Blanket of uniform thickness:**

$$x_r = x_3 \left(\frac{H}{h_a} - 1\right) - L_2 \qquad (1)$$

and

$$x_r = \frac{\tanh\left[L_B \sqrt{\frac{k_B}{k_f\ d\ z_B}}\right]}{\sqrt{\frac{k_B}{k_f\ d\ z_B}}} = \frac{\tanh c_B L_B}{c_B} \tag{2}$$

Various combinations of $\frac{z_B}{k_B}$ and L_B can be determined from Fig. 2 for x_r. The best combination of z_B and L_B is that along the x_r curves near the dashed line. After selecting L_B

$$\frac{z_B}{k_B} = \frac{1}{k_f\ d\ {c_B}^2} \tag{3}$$

z_{bL} should always be at least 3 ft.

b. Blanket of triangular section:(1)

$$\frac{z_B}{k_B} = \frac{L_B\ x_r}{\left(\frac{L_B}{x_r} - 1\right) k_f\ d} \tag{4}$$

where z_B = thickness of blanket at levee

L_B = length of triangular blanket.

Case II. Existing Natural Uniform Top Stratum and Blanket from Levee to River

$$x_r = x_3 \left(\frac{H}{h_a} - 1\right) - L_2 \tag{1}$$

$$x_r = \frac{\tanh\left[L_1 \sqrt{\frac{k_{Bb}}{k_f\ d\ z_{Bb}}}\right]}{\sqrt{\frac{k_{Bb}}{k_f\ d\ z_{Bb}}}} = \frac{\tanh c_{Bb} L_1}{c_{Bb}} \tag{2a}$$

From Fig. 2, obtain z_{Bb} for L_1, then

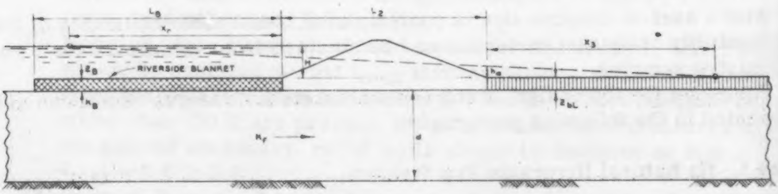


Fig. 1. Nomenclature for designing riverside blankets. No natural riverside top stratum

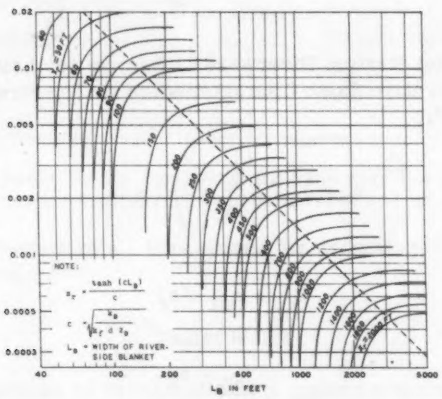


Fig. 2. Values of L_B and c for x_r. Finite length of riverside blanket on pervious substratum

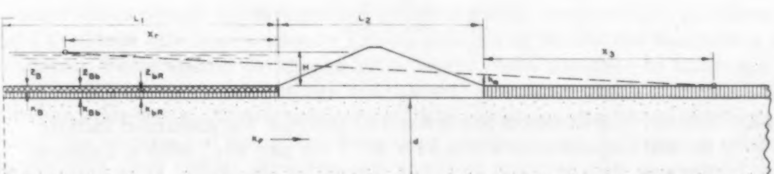


Fig. 3. Nomenclature for designing riverside blankets. Natural uniform top stratum from levee to river

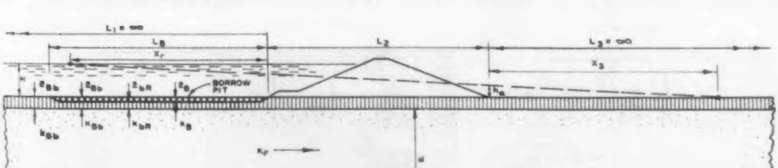


Fig. 4. Nomenclature for designing riverside blankets in borrow pits. Natural top stratum riverward of levee infinite and same as top stratum and blanket in borrow pit

$$\frac{z_{Bb}}{k_{Bb}} = \frac{1}{k_f \ d \ c_{Bb}^2} \tag{3a}$$

Case III. Natural Top Stratum Riverward of Borrow Pit Assumed Infinite ($L_1 > 2000$ ft) and To Have Same Characteristics as Top Stratum and Uniform Blanket in Borrow Pit.

$$x_r = x_3 \left(\frac{H}{h_a} - 1 \right) - L_2 \tag{1}$$

Assume

$$z_{Bb} = z_{bR} + z_B$$

Assume

$$k_{Bb} = \frac{z_{bR} + z_B}{\frac{z_{bR}}{k_{bR}} + \frac{z_B}{k_B}}$$

$$\frac{z_B}{k_B} = \frac{x_r^2}{k_f d} - \frac{z_{bR}}{k_{bR}} \tag{5}$$

Case IV. Natural Top Stratum Riverward of Borrow Pit Assumed Infinite ($L_1 > 2000$ ft) and Impervious ($k < 0.05 \times 10^{-4}$ cm per sec) With a Uniform Blanket in Borrow Pit

$$x_r = x_3 \left(\frac{H}{h_a} - 1 \right) - L_2 \tag{1}$$

and

$$x_r = \frac{1}{\sqrt{\frac{k_B}{k_f \ d \ z_B}} \quad \tanh \left[L_B \sqrt{\frac{k_B}{k_f \ d \ z_B}} \right]} = \frac{1}{c_B \ \tanh \ c_B \ L_B} \tag{6}$$

The value of c_B or $\sqrt{\frac{k_B}{k_f \, d \, z_B}}$ can be obtained from Fig. 6 for any given width of borrow pit. From c_B

$$\frac{z_B}{k_B} = \frac{1}{k_f \ d \ c_B^2} \tag{3}$$

According to Bennett(1) a triangular-shaped blanket will tend to be about 25 per cent more efficient for the same length and volume of material as a uniform blanket of constant thickness. Triangular blankets are desirable where long blankets are contemplated but probably would not be used in

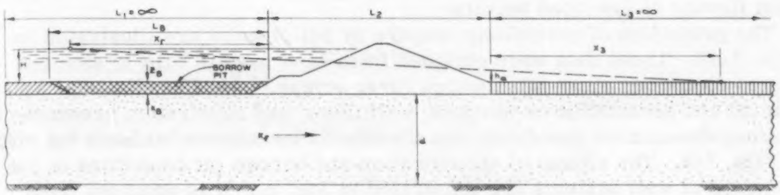


Fig. 5. Nomenclature for designing riverside blankets. Top stratum riverward of borrow pit infinite and impervious

narrow riverside borrow pits. Procedures for designing triangular blankets are given in reference.(1)

Relief Wells

The primary purpose of relief wells is to reduce artesian pressures above the ground surface which otherwise would cause formation of sand boils and possibly subsurface piping. Relief wells also intercept and provide controlled outlets for seepage which otherwise would emerge uncontrolled landward of the levee.

Relief wells should be designed to penetrate into the principal pervious strata to obtain efficient pressure relief, especially where the foundation is stratified. Wells should be spaced sufficiently close together to intercept seepage and reduce to safe values hydrostatic pressure which otherwise would act beyond the wells. Wells must offer little resistance to water flowing through the screen and out of the well; they must prevent infiltration of sand into the well after initial development; and they must be able to resist the deteriorative action of water, soil, and bacteria.

Disadvantages of relief wells are that they require periodic inspection and maintenance, they must be protected from back flooding, and they increase the total quantity of seepage about 20 to 40% depending on conditions. However, these disadvantages can partially be overcome by providing a suitable

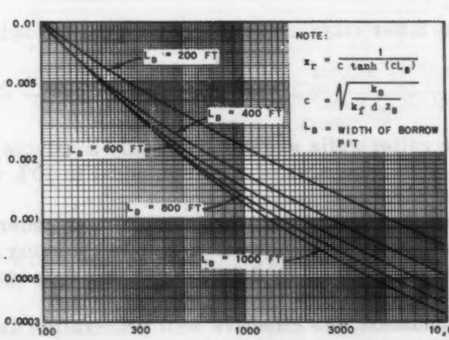


Fig. 6. Values of L_B and c for x_r. Top stratum riverward of borrow pit impervious and infinite in extent

well guard, check valve and rubber gasket, and standpipe to prevent the wells from flowing at low flood heights.

The principles of controlling seepage by relief wells are illustrated by Figs. 7-10. These data were obtained from sand models built to simulate typical conditions along Mississippi River levees.(4,6) The effects of well spacing and penetration on seepage, well flows, and substratum pressures in a homogeneous sand foundation are illustrated for various landside top strata in Figs. 7-9. The effects of stratification and borrow pit conditions on the operation of well systems are illustrated in Fig. 10. It is apparent from Fig. 10 that wells should penetrate the more pervious strata of the substratum in order to relieve substratum pressures efficiently. From Fig. 9 it is seen that relief wells increase the total quantity of seepage somewhat, although they materially reduce the natural seepage through the landside top stratum (e.g., wells on 150-ft spacing increased the total seepage 25% but decreased the seepage emerging through the top stratum by 75%).

Pertinent factors to be considered in the design of well systems are well radius, well spacing, depth and permeability of the foundation, stratification of the foundation, distance to the effective source of seepage, characteristics of the landside top stratum, net head on the levee, and degree of pressure relief or seepage interception desired.

Design of the Well

The design of the well itself consists of the selection of type and length of riser pipe and screen, design of the gravel filter, and design of relief well appurtenances. Treated wood-stave riser and screen are economical and noncorrosive, and are recommended for relief wells. The uppermost 10 to 15 ft of the riser pipe should be surrounded by concrete backfill to insure against decay resulting from fluctuations in ground-water level. To prevent filter gravel from entering the well and to minimize screen entrance head losses, the slots in the well screen must have adequate area and yet be of such size as to prevent movement of filter through the screen after development of the well (see criteria below).

$$\frac{(\text{Min})D_{85}\ \text{Filter}}{\text{Slot width}} \geqq 1.2 \quad \text{or} \quad \frac{(\text{Min})D_{85}\ \text{Filter}}{\text{Hole diam}} \geqq 1.0$$

The gradation of the filter must also comply with the following criteria:

$$\frac{(\text{Max})D_{15}\ \text{Filter}}{(\text{Min})D_{85}\ \text{Sand}} \leqq 5.0 \quad \text{and} \quad \frac{(\text{Min})D_{15}\ \text{Filter}}{(\text{Max})D_{15}\ \text{Sand}} \geqq 4$$

Wooden screens for relief wells are commercially available with 3/16-x 3-1/4-in. slots and with open area of the slots equal to 10% of the circumferential area of the screen.

Wells in the alluvial valley of the Mississippi River generally have an inside diameter of 8 in. in order to have an adequate carrying capacity without excessive head loss in the well. An 8-in. well with a 6-in. gravel filter has an effective radius of about 0.8 to 1.0 ft.

In a stratified foundation, the effective well penetration usually differs from that computed from the ratio of the length of well screen to total thickness of the aquifer. The effective screen penetration W of a well screen length $\overline{W}$ in a stratified foundation can be determined in the following manner.

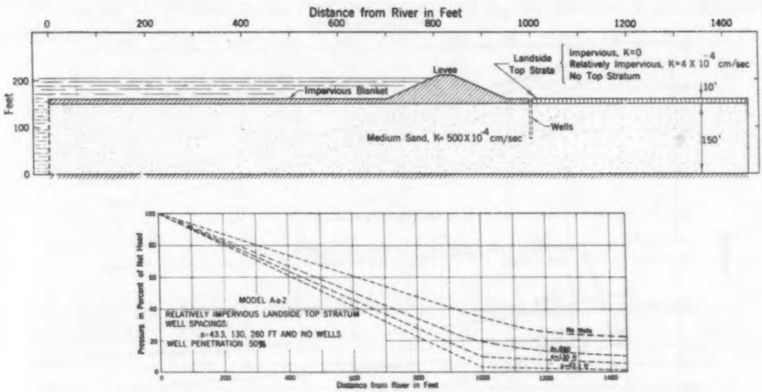


Fig. 7. Hydraulic grade line beneath top stratum with and without relief wells. Homogeneous sand foundation and relatively impervious landside top stratum (Model A)

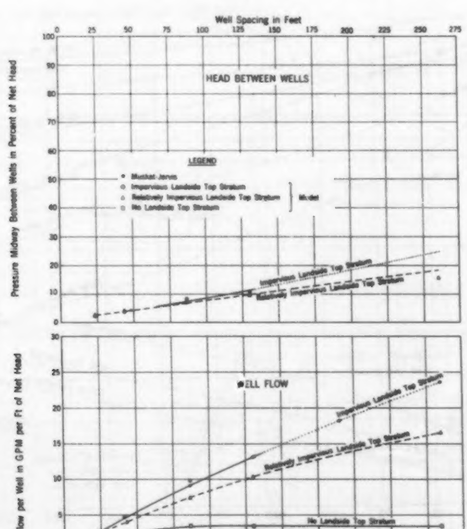


Fig. 8. Well flows and landside substratum pressures. Homogeneous sand foundation, various landward top strata (Model A). Well penetration, 50%

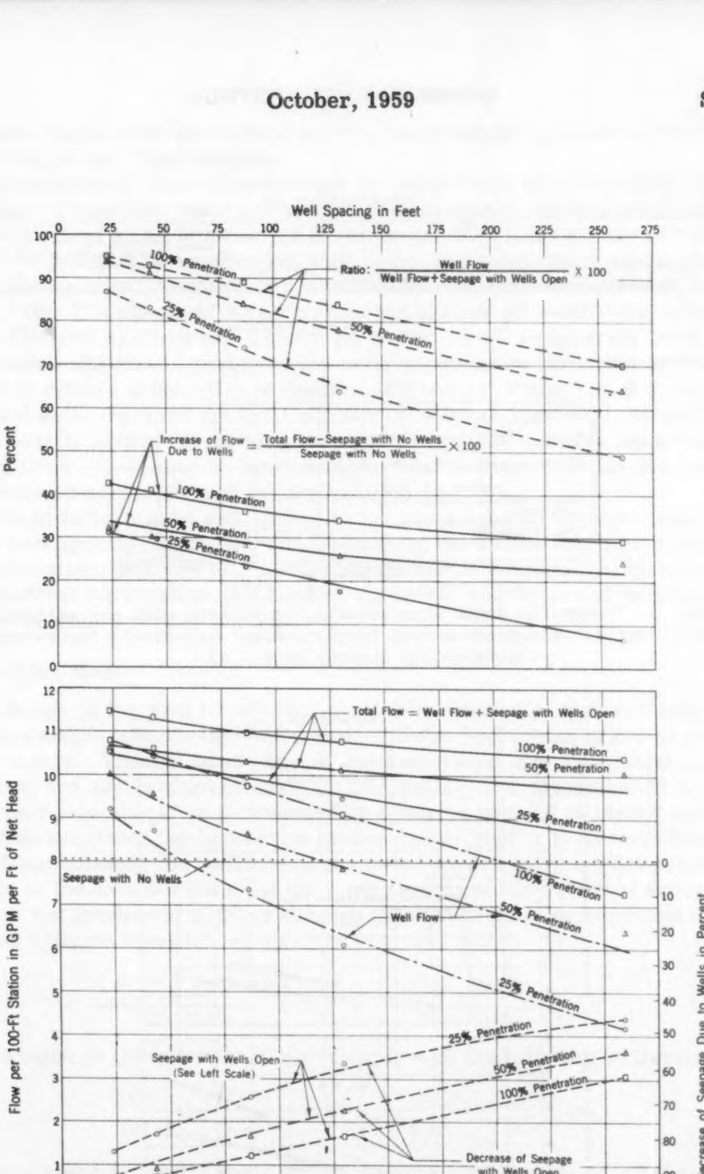


Fig. 9. Well flow and seepage. Homogeneous sand foundation and relatively impervious landside top stratum (Model A)

Each stratum of the pervious substratum with thickness d_n and horizontal and vertical permeability coefficients k_{H-n} and k_{V-n} can be transformed into an isotropic layer of thickness $\bar{d}_n$ and permeability $\bar{k}_n$ by means of the following equations:

$$\bar{d}_n = d_n \sqrt{\frac{k_{H-n}}{k_{v-n}}}$$

$$\bar{k}_n = \sqrt{k_{H-n}\, k_{v-n}}$$

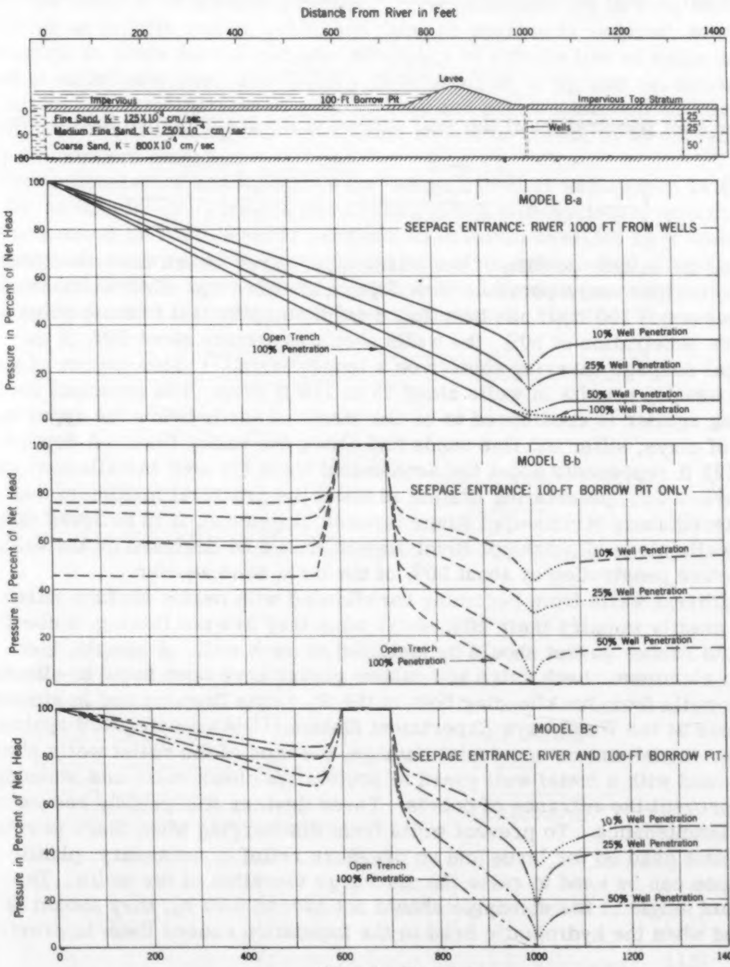


Fig. 10. Hydraulic grade line beneath top stratum with relief wells and various seepage entrances. Stratified sand foundation and impervious landside top stratum (Model B)

The thickness of the transformed, homogeneous, isotropic foundation $\bar{D}$ is

$$\bar{D} = \sqrt{\sum (d_n \; k_{H-n}) \; \sum (d_n / k_{v-n})}$$

and the effective permeability of the transformed foundation $\bar{K}$ is

$$\bar{K} = \sqrt{\frac{\sum (d_n \; k_{H-n})}{\sum (d_n / k_{v-n})}}$$

The effective well screen penetration W into the transformed foundation is

$$\bar{W} = \frac{\sum_0^{\bar{W}} d_n \; k_{H-n}}{\bar{K}}$$

The per cent penetration of the well screen in the transformed foundation is

$$\left(\frac{\bar{W}}{\bar{D}}\right) \% = \frac{100 \sum_0^{\bar{W}} d_n \; k_{H-n}}{\bar{K} \; \bar{D}} = \frac{100 \sum_0^{\bar{W}} d_n \; k_{H-n}}{\sum_0^{\bar{D}} d_n \; k_{H-n}}$$

Along the middle portion of the Mississippi River where the substratum tends to become more pervious with depth and where the effective thickness averages about 100 ft, it has been found from pumping tests that to achieve an effective penetration of 50%, the wells should penetrate about 60% of the principal seepage carrying aquifer on a length basis.(7) This degree of penetration usually results in wells about 75 to 110 ft deep. The principal seepage carrying aquifer is considered to be the strata of sands below the upper top strata of clays, silts, and fine sands and above the valley floor. A depth of about 125 ft represents about the economical limit for well installation; and, therefore, a 50% penetrating system is about the practical maximum that can be achieved along Mississippi River levees. In general, it is believed that relief wells along Mississippi River levees should be designed on the basis of an effective penetration of about 50% of the main sand aquifer.

To prevent wells from becoming backflooded with muddy surface water (which greatly impairs their efficiency) when they are not flowing, a check valve and rubber gasket should be installed on each well. A simple, inexpensive aluminum check valve and rubber gasket have been found to effectively protect wells from backflooding both in the St. Louis District and in simulated field tests at the Waterways Experiment Station.(7) As a safeguard against animals, vandalism, or accidental damage, the tops of the relief wells should be provided with a metal well guard to protect the check valve and standpipe and to prevent the entrance of debris. These devices will greatly reduce required maintenance. To prevent wells from discharging when there is relatively little head on the levee and no pressure relief is necessary, plastic standpipes can be used to raise the discharge elevation of the wells. The maximum height of the standpipe should not exceed 0.25 h_a; they should be removed when the hydrostatic head in the foundation causes them to overflow.

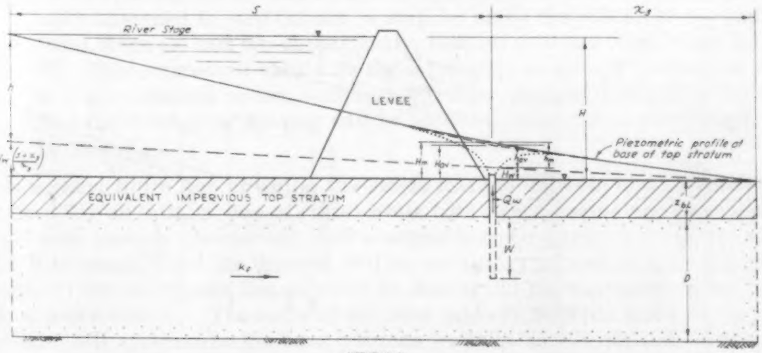
Design Formulas

Formulas for designing relief wells have been developed from theoretical and model studies,(2) but until recently these formulas were limited to fully

penetrating wells with either an impervious or leaking top stratum, or partially penetrating wells with an impervious top stratum. For conditions encountered along the Lower Mississippi River, formulas are required for designing partially penetrating wells with a leaking top stratum. Recently, electrical analogy model studies have been conducted by the Corps of Engineers, U. S. Army, to obtain design curves for partially penetrating wells with a leaking top stratum; design curves and procedures contained herein are based on data given in reference (3).

The design of a well system consists essentially of determining the spacing and penetration of wells that will reduce the substratum pressure h_o at the toe of the levee to an allowable head h_a. The wells spacing is first determined assuming an infinite line of wells, and then the spacing is reduced, where necessary, to allow for the reduced efficiency of a finite line of wells as compared to an infinite line. For given values of h_a, H, s, k_f, and x_3, there are any number of combinations of well spacing and penetration that will suffice; the final selected spacing and penetration should be based, to an extent, on the most economical design.

The nomenclature and equations for design of relief wells given in Fig. 11 are for an infinite line of wells penetrating into a homogeneous, isotropic (either natural or transformed) pervious substratum overlain by a leaking top stratum. To determine the required spacing of an infinite line of wells of given W and W/D, it is necessary to utilize the following procedure of successive trials, and the nomograph for well design in Fig. 12. The required



NOTATIONS

h = EFFECTIVE NET HEAD ON THE WELL SYSTEM
H = TOTAL NET HEAD ON THE LEVEE
h_a = ALLOWABLE HEAD BENEATH TOP STRATUM
H_w = TOTAL HEAD LOSS IN WELL INCLUDING ELEVATION HEAD LOSS
H_m = NET HEAD BENEATH TOP STRATUM MIDWAY BETWEEN WELLS
H_{av} = NET HEAD IN THE PLANE OF WELLS
h_{av} = NET AVERAGE HEAD IN THE PLANE OF WELLS ABOVE H_w
h_m = NET HEAD BENEATH TOP STRATUM MIDWAY BETWEEN WELLS ABOVE H_w
θ_{av} = AVERAGE UPLIFT FACTOR
θ_m = MIDPOINT UPLIFT FACTOR
ΔM = NET SEEPAGE GRADIENT TOWARD THE WELL LINE
a = WELL SPACING
r_w = EFFECTIVE WELL RADIUS
W = EFFECTIVE LENGTH OF WELL SCREEN
D = EFFECTIVE THICKNESS OF PERVIOUS SUBSTRATUM
Q_w = FLOW FROM A SINGLE WELL

FORMULAS

$$h = H - H_w \left(\frac{s + x_3}{x_3}\right) \quad (7)$$

$$h_{av} = \frac{h\,\theta_{av}}{\frac{s}{a} + \left(\frac{s + x_3}{x_3}\right)\theta_{av}} \quad (8)$$

$$H_{av} = H_w + h_{av} \quad (9)$$

$$h_m = h_{av}\frac{\theta_m}{\theta_{av}} = \frac{h\,\theta_m}{\frac{s}{a} + \left(\frac{s + x_3}{x_3}\right)\theta_{av}} \quad (10)$$

$$H_m = H_w + h_m \quad (11)$$

$$\Delta M = \frac{H - H_{av}}{s} - \frac{H_{av}}{x_3} \quad (12)$$

$$Q_w = \frac{h\,k_f\,D}{\frac{s}{a} + \left(\frac{s + x_3}{x_3}\right)\theta_{av}} = a\,\Delta M\,k_f\,D \quad (13)$$

$$h_{av} = a\,\Delta M\,\theta_{av} \quad (14)$$

$$h_m = a\,\Delta M\,\theta_m \quad (15)$$

Fig. 11. Nomenclature and formulas for design of relief wells

well spacing is affected by hydraulic head losses in the well; these losses which consist of screen entrance loss, friction loss, and velocity head loss can be estimated for an 8-in. ID wood-stave well from Fig. 13.

The procedure for computing the well spacing is outlined below:

a. Compute h_a from $h_a = i_o z_t$.
b. Assume that $H_{av} = h_a$ and compute ΔM from Eq. (12).
c. Assume a well spacing a and compute Q_w from Eq. (13).
d. Estimate H_w for the above Q_w and W/D by means of Fig. 13.
e. Compute h_{av} from Eq. (9).
f. Substitute the above values of h_{av} and ΔM in Eq. (14) and solve for θ_{av} for various values of a.
g. Find θ_{av} from Fig. 12 for the values of a used in step (f) and the corresponding a/r_w and D/a values.
h. The first trial well spacing is that of value a for which θ_{av} from step (f) = θ_{av} from step (g).
i. Find θ_m from Fig. 12 for the first trial well spacing and the corresponding values of a/r_w and D/a.
j. If $\theta_{av} > \theta_m$, repeat procedure steps (c) to (i) inclusive using the first trial well spacing in lieu of the spacing originally assumed in step (c),

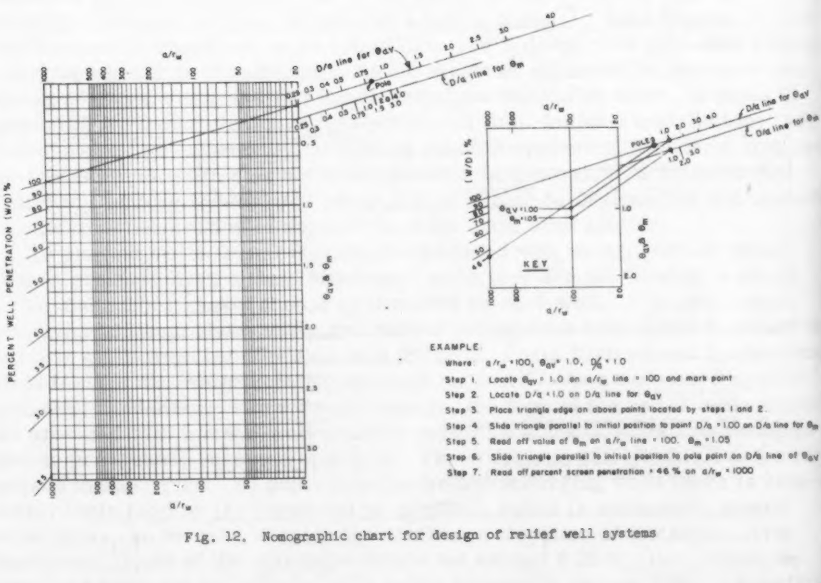


Fig. 12. Nomographic chart for design of relief well systems

and determine the second trial well spacing. The above procedure should be repeated until relatively consistent values of a are obtained on two successive trials, although usually the second trial spacing is sufficiently accurate. If in procedure step (j), $\theta_{av} < \theta_m$, then the following procedure should be used:

k. Assume that $H_m = h_a$ and compute Q_w from Eq. (13), using the value of ΔM previously obtained in step (b) and the first trial well spacing previously found from step (h).
l. Estimate H_w from Q_w of step (k) and W/d, by means of Fig. 13.
m. Compute h_m from Eq. (11) from H_w obtained in step (l).
n. Compute h_{av} from Eq. (10) using h_m from step (m) and θ_{av} and θ_m from steps (h) and (i), respectively.
o. From the above values of h_{av} and H_w, compute H_{av} from Eq. (9).
p. Compute ΔM from Eq. (12) using H_{av} from step (o).
q. Substitute the above values of h_m and ΔM in Eq. (15) and solve for θ_m for various values of a.
r. Find θ_m from Fig. 12 for the values of a used in step (q) and the corresponding a/r_w and D/a values.
s. The second trial well spacing is that value of a for which θ_m from step (q) = θ_m from step (r).
t. Find θ_{av} from Fig. 12 for the second trial well spacing and the corresponding values of a/r_w and D/a.
u. Determine the third trial well spacing by repeating steps (k) to (t) inclusive, using the second trial well spacing in lieu of the spacing originally assumed in step (k) and in step (n) using the values of θ_m and θ_{av} from steps (s) and (t), respectively, instead of those from steps (h) and (i). This procedure should be repeated until relatively consistent values of a are obtained on two successive trials. Normally it will be found that the third trial spacing will be sufficiently accurate for design purposes.

In a short finite line of wells, the heads midway between wells exceed those obtained for an infinite line of relief wells both at the center and near the ends of the well system. Numerous well systems may be fairly short (less than 1200 ft in length), and for these it will be necessary to reduce the well spacing computed for an infinite line of wells so that heads midway between wells will not be more than h_a. The ratio of the head midway between wells at the center of finite well systems to the head between wells in an infinite line of wells is shown in Fig. 14 for various well spacings and exit lengths. The spacing of wells in a finite line should be the same as that required in an infinite line of wells to reduce the head midway between wells to h_a divided by ratio of $\frac{H_{m_N}}{H_{m_\infty}}$ from Fig. 14. In any finite line of wells of constant penetration and spacing, the head midway between wells near the ends of the system exceeds that at the center of the system. Thus, at the ends of both short and long well systems, the wells should generally be made deeper to provide additional penetration of the pervious substratum so as to obtain the same head reduction as in the central part of the well line.

After the well spacing for a given reach of levee has been determined, the location of each well should be checked in the office and in the field and adjusted where necessary so that the wells will be located at critical seepage spots and will fit natural topographic features.

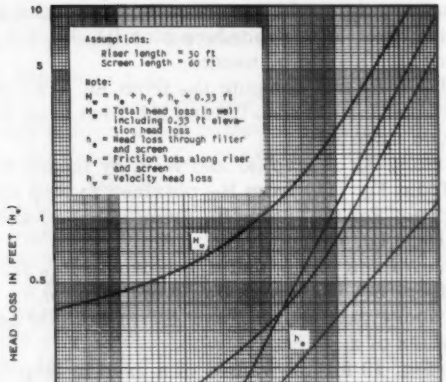


Fig. 13. Hydraulic head losses in 8-in. ID wood-stave well with 6-in. gravel filter

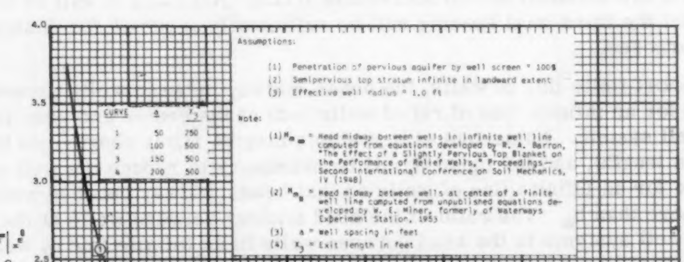


Fig. 14. Ratio of head midway between wells at center of a finite well system to head midway between wells in an infinite line of wells

A set of design curves generally applicable for designing relief well systems for levees along the Lower Mississippi River has been developed for average foundation conditions and for distances to the effective source of seepage of 500, 1000, 1500, and 2000 ft[8] (see Fig. 15 for s = 1000 ft). The curves are for wells with an effective penetration of 50%, r_w = 1 ft and D = 100 ft. The heads midway between wells in per cent are based on θ_{av} or θ_m, whichever is greater, and are valid for H = about 20 to 35 ft. The well flows are based on $k_f = 1250 \times 10^{-4}$ cm/sec, or about the average for sites above L'Argent, La., as the most critical reaches of levee as regards seepage generally exist upstream of this point. Where k_f is not equal to 1250×10^{-4} cm per sec and/or $D \neq 100$ ft, Q_w/H can be determined from the curves in Fig. 15, and then multiplied by $\dfrac{k_f D}{125{,}000}$ where k_f is in 10^{-4} cm per sec units and D is in feet.

Landside Seepage Berms

A landside berm can be used to control seepage by increasing the thickness of the top stratum immediately landward of the levee so that the weight of berm plus top stratum is sufficient to resist uplift pressures beneath the top stratum. A properly designed berm will be of such width that the excess head beneath the top stratum at the toe of berm is no longer critical, or the area of possible rupture of the top stratum is removed a sufficient distance from the levee as to no longer endanger it. A landside berm also affords some protection against possible sloughing of the landside slope of the levee as a result of the seepage.

Berms can be used to control seepage efficiently where the landside top stratum is relatively thin and uniform or where no landside top stratum is present. However, they are not very feasible where the top stratum is relatively thick and high uplift pressures develop as the thickness and width of berm required to reduce upward gradients to those recommended herein would be excessive. Where the landside top stratum is irregular, berms will force the point of seepage emergence farther from the levee, but concentrations of seepage and sand boils may still develop at thin spots in the top stratum at the berm toe. Where a levee is founded on thin top stratum and thick clay deposits lie a short distance landward of the levee, the seepage berm should be of sufficient width and thickness to cover the near edge of the thick clay if practicable; otherwise, the berm will tend to concentrate the seepage in the area between the berm toe and the thick clays. Seepage berms should generally have a slope of 1 on 50 or steeper to insure drainage. However, if the berm is constructed after the levee has caused the foundation to consolidate fully, a slope of 1 on 75 can be used.

Design Formulas

Berms may vary in character from impervious to completely pervious and free draining. In view of this, design formulas are presented for impervious, semipervious, and sand berms, and a completely pervious, free-draining berm. Where a landside top stratum is present, the berm should have a thickness so that i_o through the top stratum and berm at the levee toe $\leqq 0.5$, and a width if practicable so that the head beneath the top stratum at the berm toe is 0.75 z_t to 0.85 z_t. Formulas for designing landside seepage berms

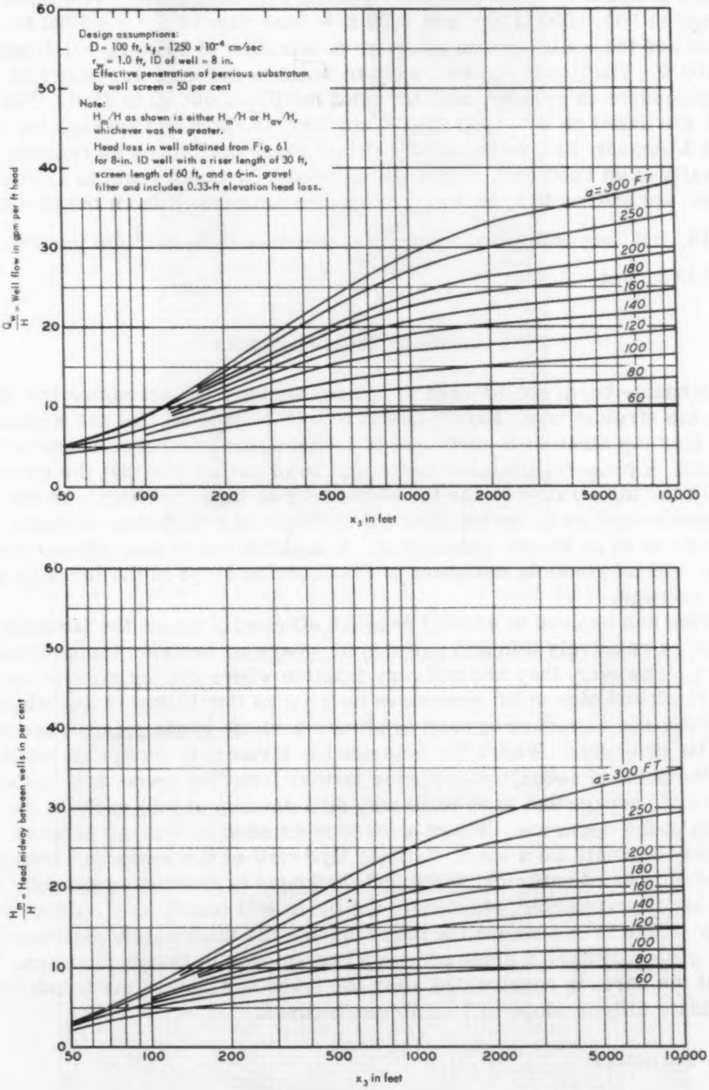
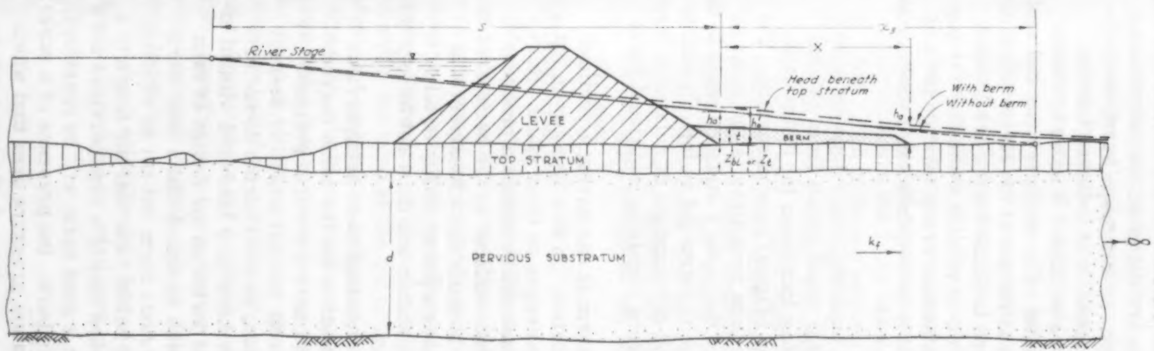


Fig. 15. Well flow and head midway between wells; s = 1000 ft



NOTATIONS

$c = \sqrt{\frac{k_{bL}}{k_f z_{bL} d}}$

γ_z' = SUBMERGED UNIT WEIGHT OF TOP STRATUM

γ_t' = SUBMERGED UNIT WEIGHT OF BERM

γ_w = UNIT WEIGHT OF WATER

h_o = HEAD AT LANDSIDE TOE OF LEVEE WITHOUT BERM = $\frac{Hx_3}{s + x_3}$

h_o' = HEAD AT LANDSIDE TOE OF LEVEE WITH BERM

i_o = ALLOWABLE UPWARD GRADIENT AT LANDSIDE TOE OF LEVEE

i_t = ALLOWABLE UPWARD GRADIENT AT TOE OF BERM

h_a = ALLOWABLE HEAD AT TOE OF BERM = $i_t z_t$

X = REQUIRED BERM WIDTH

t = REQUIRED THICKNESS OF BERM AT TOE OF LEVEE

k_t = VERTICAL PERMEABILITY OF BERM

F = FACTOR OF SAFETY AGAINST UPLIFT AT TOE OF LEVEE

Q_s = FLOW INTO BERM PER FT OF LEVEE

FORMULAS

IMPERVIOUS BERM ($k_t = 0$)

$$X_I = x_3\left(\frac{H}{h_a} - 1\right) - s$$

$$h_o' = H\left(\frac{x_3 + X}{s + x_3 + X}\right)$$

$$t = \frac{h_o' - z_t\left(\frac{\gamma_z'}{F\gamma_w}\right)}{1 + \frac{\gamma_t'}{F\gamma_w}}$$

USE $F \geq 1.5$

SEMIPERVIOUS BERM ($k_t = k_{bL}$)

$$X_{sp} = \frac{-A + \sqrt{A^2 - 24(2 + r)\left(1 + sc - \frac{H}{h_a}\right)}}{2c(2 + r)}$$

WHEREIN:

$$A = 6 + 3sc\,(r + 1)$$

$$r = \frac{i_o}{i_t}$$

$$h_o' = h_a\left[1 + cX + \left(\frac{2 + r}{6}\right)(cX)^2\right]$$

$$t = \frac{h_o' - i_o z_t}{1 + i_o}$$

SAND BERM

$$X_s = \frac{1}{3}\left(X_p + 2X_{sp}\right)$$

$$h_o' = h_a\left[1 + cX + \left(\frac{2 + r}{6}\right)(cX)^2\right]$$

$$t = \frac{h_o' - z_t\left(\frac{\gamma_z'}{F\gamma_w}\right)}{1 + \frac{\gamma_t'}{F\gamma_w}}$$

PERVIOUS BERM WITH COLLECTOR

$$X_p = x_3 \log_e\left(\frac{h_o'}{h_a}\right)$$

$$h_o' = h_o = \frac{Hx_3}{s + x_3}$$

$$t = \frac{h_o' - z_t\left(\frac{\gamma_z'}{F\gamma_w}\right)}{1 + \frac{\gamma_t'}{F\gamma_w}}$$

$$Q_s = \frac{k_f dH}{s + x_3}\left(1 - e^{\frac{-X}{x_3}}\right)$$

Fig. 16. Nomenclature and formulas for designing landside seepage berms on semipervious top stratum

overlying a semipervious top stratum are given in Fig. 16; items pertinent to the design of each type of berm are discussed in the following paragraph.

The formulas shown in Fig. 16 permit determination of the berm width and the thickness at the toe of the levee; formulas are not given for determining the required thickness at the edge of the crown, as a seepage berm theoretically tapers to zero thickness at its toe. However, it is believed that the thickness of a berm at the outer edge should be at least 1 ft so as to define the limits of the berm for maintenance purposes. For semipervious and sand berms to function as intended their thickness at the toe of the levee should not greatly exceed the computed thickness. Where landside berms are founded directly on the pervious substratum they should be of such width that the combined width of levee and berm satisfies the minimum creep ratio criteria of Bligh and Lane. These berms should preferably be constructed of sand, or as a free-draining berm.

a. Impervious berms. The presence of a landside impervious berm restricts the natural relief of pressure that would result from natural seepage through the top stratum, and thus increases the hydrostatic head at the levee toe with respect to the original ground surface. The effect of an impervious berm on substratum pressures is the same as increasing the impervious base width of the levee a distance equal to the width of the berm. An impervious berm constructed on top of relatively pervious top strata tends to cause the development of relatively large uplift pressures beneath the berm, thereby requiring additional berm thickness.

b. Semipervious berm. A semipervious berm is one having a vertical permeability equal to that of the top stratum. In this type berm, natural seepage passes through the berm and emerges on its surface. However, even this type of berm will increase the substratum pressure at the levee toe (measured above the ground surface) as the berm creates additional resistance to seepage flow. In order for a semipervious berm to function as intended, it must have a permeability equal to or greater than that of the underlying top stratum and must not be appreciably thicker than the computed thickness.

c. Sand berm. Sand berms have a slight advantage over semipervious berms in that less berm material is required for the same degree of seepage protection. A sand berm should have a vertical permeability of at least 100×10^{-4} cm per sec. Even with this permeability, seepage into the berm must emerge at its surface, as sand berms do not have sufficient capacity to conduct any appreciable flow landward without excessive internal head loss. Theoretical formulas for design of sand berms were not developed. Instead it was assumed that a sand berm would be more efficient than a semipervious berm but not as efficient as a pervious, free-draining berm (see below), and that the length of a sand berm should be intermediate between that of a semipervious and a pervious free-draining berm. Although a sand berm will be considerably more pervious than a semipervious berm, the presence of a sand berm will increase the landside substratum pressure over that which would exist without a berm, because seepage through the berm must emerge at the berm surface. As a result the dimensions of a sand berm are considered more closely related to those of a semipervious than to those of a free-draining berm.

d. Free-draining berm. A free-draining berm is one where the seepage enters the berm, is collected and discharged landward with low internal head losses in the berm. Such a berm will not affect the natural seepage flow pattern or the distribution of landside substratum pressures and, therefore, is the narrowest and thinnest of all berms required for a given foundation condition. However, for a berm to be free-draining it must be underlain by properly designed sand and gravel filters and a collector system. The sand and gravel blankets beneath the collector pipes should have a minimum thickness of 6 in. The gravel layer should be covered with 4 to 6 in. of the sand filter to prevent the overlying random soil from migrating into the gravel. The landside edge of the berm should consist of about 3 ft of random soil to protect the gravel blanket against backflooding with muddy surface water. The material above the filter blankets and collector system can be of random soil. The collector system should be capable of collecting and discharging the flow into the berm (which can be estimated from Fig. 16) with small head losses. The collector pipes should be of extra-strength vitrified clay tile, or equivalent, perforated with 1/4- or 3/8-in. holes, and should have a minimum ID of 6 in. The ends of the outfall pipes from the collector system should be of unperforated pipe and should terminate in a tee with a short vertical sleeve, rubber gasket and flat-type check valve, and an outlet guard to prevent backflooding with muddy surface water and the entrance of small animals. The discharge of the outfall pipe should be set about 4 to 6 in. above the natural ground surface.

Maximum Widths

Where the computed width of a berm required to reduce the substratum pressure at its toe to an allowable amount ($\sim 0.8\ z_{bL}$) exceeds 300 to 400 ft, the berm would not be made wider than 300 to 400 ft as it is considered that a levee would probably be safe against underseepage even with sand boils within such distances. Where the selected width of berm is less than the computed width, the head at the toe of the levee or existing berm h'_o would not be as great or t as thick as indicated by the equations in Fig. 16. For the selected berms, h'_o would be recomputed assuming an i_1 of 0.8 at the toe of the new berm and a linear piezometric grade line between the toe of the new berm and the point of effective seepage entry. The recommended thickness of the berm would be based on values of h'_o expected to develop with a berm of the selected width, whereas the original computed thickness would be based on the h'_o corresponding to a berm having a width equal to the computed X. The estimated seepage flow Q_s can be determined from the h'_o corresponding to the selected berm.

The final selection of a berm should be based on the availability of borrow materials and the relative cost of each type berm.

Drainage Trenches

Drainage trenches can be used to control underseepage where the top stratum is thin and the pervious foundation is shallow so that the trench can be built to penetrate the aquifer substantially. Where the pervious foundation is deep, a drainage trench of any reasonable depth would attract only a small

portion of underseepage, its effect would be local, and detrimental underseepage would bypass the trench. Because of the depth of the pervious substratum along Mississippi River levees, drainage trenches are not considered feasible for these levees. However, they may possibly be applicable to levees along other rivers.

The existence of moderately impervious strata or even stratified fine sands between the bottom of the drainage trench and the underlying main sand aquifer will render ineffective or decrease the efficiency of a drainage trench. Seepage into a drainage trench, where the top stratum landward of the levee consists of an impervious or relatively impervious blanket, may be computed from the graphs and formulas given in Fig. 17. The maximum head landward of the drainage trench may also be computed from these graphs. It is pointed out that the formulas and graphs(3) shown in this figure are applicable only for homogeneous, isotropic, pervious foundations, and for an impervious top stratum landward of the drain.

If $k_H > k_V$, as is usually the case for alluvial sands, flow to and head landward of a drainage trench can be estimated from Fig. 17 after the pervious substratum is transformed to a homogeneous, isotropic formation using $\bar{k}$ and $\bar{d}$ for k_f and d, respectively, where $\bar{k}_f = \sqrt{k_H k_V}$ and $\bar{d} = d\sqrt{k_H/k_V}$. A ratio of $k_H/k_V = 4$ is suggested for the Middle and Lower Mississippi River Valley.

If the top stratum landward of the drainage trench has a certain degree of perviousness, seepage into the trench, and the maximum head landward of the trench, will be somewhat less than that computed from Fig. 17. Therefore, designs based on Fig. 17 will be slightly on the conservative side if the top stratum landward of the trench is semipervious.

Where there is no top stratum landward of the levee, seepage flow into the drainage trench and beyond can be estimated from flow net analyses.

If the pervious aquifer is highly stratified, or if strata of either clay, silt, or fine sand exist below the bottom of the trench, seepage may bypass the

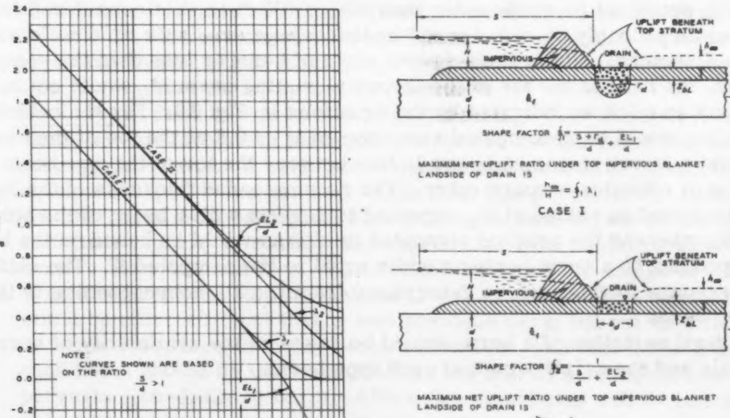


Fig. 17. Formulas and design curves for drainage trenches

drain and emerge landward of the drain, thereby defeating its purpose. For such cases, other methods of seepage interception are more reliable and efficient. If the trench is underlain by either impervious or semipervious strata, the formulas and graphs shown in Fig. 17 are no longer applicable.

An example of seepage computations and design of a drainage trench are shown in Fig. 18.

The filters comprising the drainage layers should be designed in accordance with the following filter criteria:

$$\frac{D_{15}\text{ sand blanket}}{D_{85}\text{ foundation sand}} < 5\ ;\quad 4 < \frac{D_{15}\text{ sand blanket}}{D_{15}\text{ foundation sand}}$$

$$\frac{D_{15}\text{ gravel blanket}}{D_{85}\text{ sand blanket}} < 5\ ;\quad 4 < \frac{D_{15}\text{ gravel blanket}}{D_{15}\text{ sand blanket}}$$

In addition the filter gravel around the perforated collector pipe should comply with the following criteria:

$$\frac{D_{85(min)}\text{ gravel}}{\text{Perforation}} > \begin{matrix}1.0(\text{holes})\\1.2(\text{slots})\end{matrix}$$

The collector pipe for a drainage trench should be made of corrosion-resistant material and should be perforated with 1/4-in. holes. The collector and riser pipes should have adequate capacity to carry the flow to the surface with less than 0.5-ft hydraulic head loss. The head loss should be computed on the basis of maximum full flow through 1/6 the length of collector pipe between risers, the tee connection, and the riser pipe. The riser should be of solid pipe, and should be set about 4 in. above the finished ground surface. The top of the riser should be provided with a rubber gasket and check valve to prevent flooding of the collector pipe and filters with muddy surface water. The top of the riser pipe may be provided with a low standpipe to prevent flow from the drainage trench at relatively low river stages on the levee. Maximum height of these standpipes should not exceed 1/4 h_c. Of course, such standpipes should be removed when they begin to overflow. The top of the riser or outlet should also be protected with a metal guard.

Cutoffs

Where practicable, the most positive method of underseepage control is to cut off all seepage beneath a levee by means of an impervious barrier which will eliminate both excess substratum pressures and the problem of seepage water landward of the levee. However, completely cutting off pervious strata 80 to 200 ft deep along extensive reaches of levees is not economically feasible. The installation of partially penetrating cutoffs will not reduce seepage and excess pressures significantly unless the cutoff penetrates 95% or more of the pervious aquifer.(5) However, shallow cutoffs along the riverside toe of levees are feasible where necessary to cut off relatively thin layers of either natural levee or crevasse sands which lie immediately beneath the base of the levee and are in turn underlain by more impervious strata.

Mathematical formulas for determining seepage flow and heads for partial cutoffs in a homogeneous foundation are given in Fig. 19. The hydraulic grade line beneath and landward of a levee underlain by a homogeneous foundation with and without partial cutoffs is illustrated in Fig. 20. This top stratum had

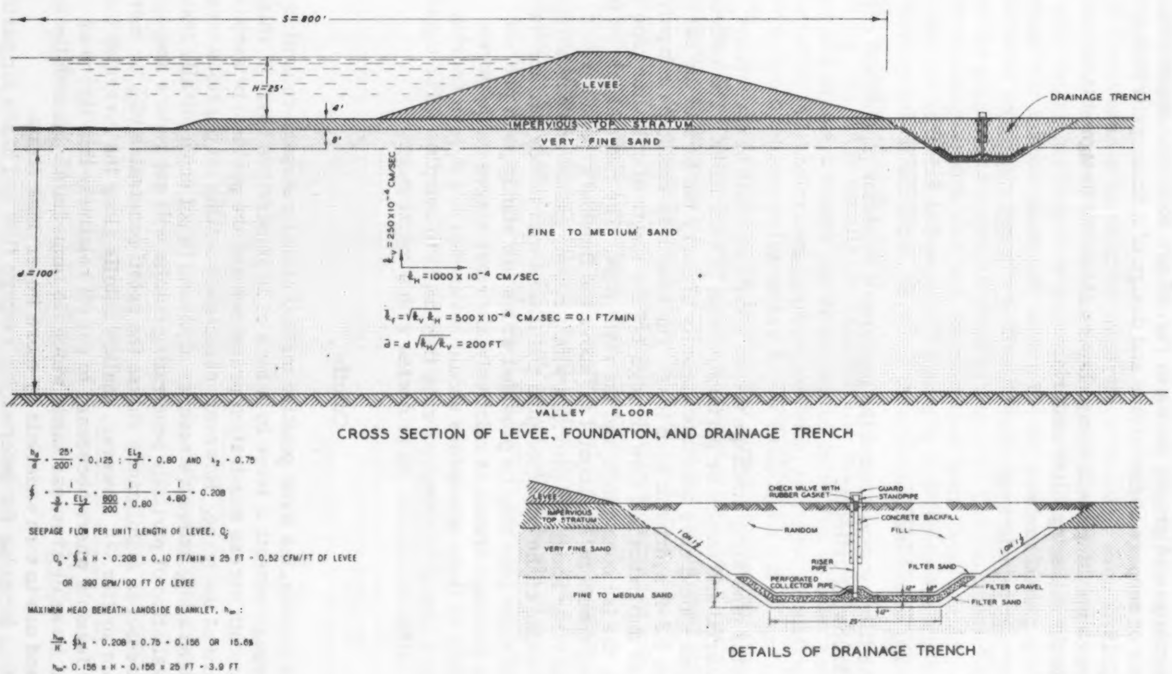


Fig. 18. Example of design of a drainage trench

Fig. 19. Formulas for determination of seepage flow and heads for partial cutoffs

such characteristics that without any cutoff the excess head at the toe of the levee was 38% of the total head. A 50% cutoff reduced the seepage flow approximately 5% and the excess head from 38% to 37%. Thus it may be seen that partial cutoffs of any practicable depth into a homogeneous aquifer have little effect in reducing seepage flow or substratum pressures landward of a levee.

This and other model studies described in reference (5) showed that partial cutoffs will not significantly reduce the amount of seepage passing beneath a levee or the excess head landward of a levee during high water. Whether or not partial cutoffs would prevent undermining of a levee as a result of piping is not known. If such a pipe developed to within a short distance of a partial riverside cutoff, there would be a good possibility that the levee might collapse into the underground cavern and cause a crevasse in spite of the partial cutoff.

Sublevees

A landside sublevee can be used to control seepage by storing water over an area to provide a counterweight against excess head beneath the top stratum in the subleveed area. Sublevees can be used to control seepage where the landside top stratum is relatively thin, and in low areas where seepage normally ponds. A disadvantage of sublevees is that if sand boils occur within the subleveed area, they may be difficult to detect or observe, and may not readily be given emergency treatment, if needed. Control of seepage by sublevees requires proper manipulation of water levels in the sublevee basins during a high water. Controlling underseepage by means of substandard sublevees is potentially hazardous as failure of a sublevee when full of water would result in losing the counterhead at a critical time.

A sublevee basin should be of sufficient width to insure that the head at the landside edge of the sublevee is not excessive and the overflow spillway should

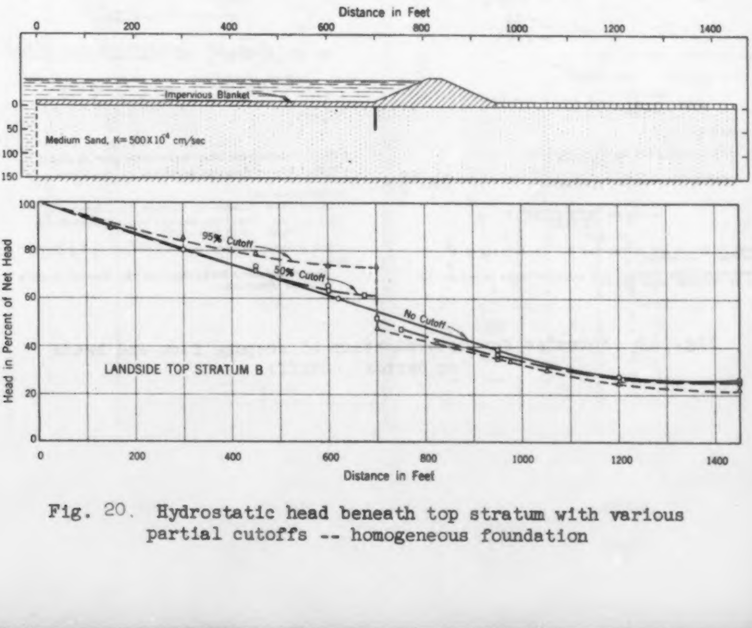


Fig. 20. Hydrostatic head beneath top stratum with various partial cutoffs -- homogeneous foundation

be set at such a height that the net excess head at the toe of the levee is not more than the allowable head.

The required width X of the sublevee basin can be approximated from the equation for the width of a sand berm in Fig. 16. Similarly the head h'_o at the toe of the levee measured above the ground surface with a sublevee basin can be estimated from the corresponding equation for h'_o with a sand berm of width X. The height of water t in the sublevee basin should be such that

$$t = h'_o - i_o z_t \tag{16}$$

where $i_o z_t$ is the allowable head at the toe of the levee. The crest of the overflow weir should be at the elevation of the required water surface in the sublevee basin, and should have sufficient length to pass both the design seepage flow Q'_s in cfs into the basin plus runoff from rainfall Q_r. The total discharge Q_T over the weir can be expressed as

$$Q_T = Q'_s + Q_r$$

wherein:

$$Q'_s = \frac{k_f\, d\, L_s\, (H - t/2)}{s + x_3} \left[1 - e^{-\frac{X}{x_3}} \right] \tag{17}$$

$$Q_r = \frac{R\, I\, L_s\, X'}{43560} \text{ in cfs} \tag{18}$$

The sublevee should have a height such that a freeboard of 1 ft will exist above the water surface in the sublevee basin with the overflow weir discharging at a rate of Q_T.

SUMMARY

It is recognized that the design procedures and formulas shown for the various methods for controlling underseepage and excess hydrostatic pressures are approximate but can serve as a useful guide to the initiate. The design formulas for the various soil conditions for the several methods strongly emphasize the importance of having adequate field soil investigations from which longitudinal and transverse profiles can be plotted, thus a proper design of a given control method can be fitted to the specific soil condition. The selection of a particular method of underseepage control will take cognizance of initial and maintenance costs and safety.

ACKNOWLEDGMENT

The seepage investigations described in this paper were conducted by the Waterways Experiment Station with the assistance of the Memphis, Vicksburg, and New Orleans Districts, CE, under the general direction of the Mississippi River Commission. The authors actively participated in the study from its inception. Mr. R. I. Kaufman, Jr. Member ASCE, assisted with analysis of the data.

APPENDIX A

Notations

A	Surface area in which seepage emerging landward of a levee is measured
a	Well spacing
c	A constant for natural top stratum where $c = \sqrt{\frac{k_b}{k_f z_b d}}$
c_B	Constant for riverside blanket
c_{Bb}	Constant for riverside blanket and natural riverside top stratum
D	Thickness of pervious substratum
$\bar{D}$	Transformed thickness of pervious substratum
D_{15}	Effective grain size, 15 per cent of grains smaller than stated size
d	Effective thickness of pervious substratum. Depth of cutoff in formulas for partial cutoffs
$\bar{d}$	Transformed total thickness of pervious substratum
$d_a, d_b \ldots, d_n$	Thickness of each stratum comprising pervious substrata
$\bar{d}_n$	Transformed thickness of each stratum comprising pervious substrata
EL_1, EL_2	The extra length of pervious substratum corresponding to the increased resistance to flow into a drainage trench as compared to flow into a fully penetrating, open, vertical, drainage face at the location of the trench
e	Void ratio
F	Factor of safety against uplift at landside toe of levee
H	Total net head on levee, or height of flood stage above average low-ground surface, or tailwater, landward of levee
H_{av}	Average head (net) in plane of relief wells
H_m	Head (net) beneath top stratum midway between relief wells
$H_{m\infty}$	Head (net) beneath top stratum midway between wells in an infinite line of wells
H_{m_N}	Head (net) beneath top stratum midway between wells at center of a finite line of wells
H_w	Total head loss in a well including elevation head loss
h	Effective (net) head acting on a line of relief wells
h_a	Allowable (net) head beneath landside top stratum
h_{av}	Average head (net) in plane of relief wells measured above H_w

h_c	Maximum possible (net) head beneath top stratum; head at which upward gradient through top stratum is equal to critical gradient
h_e	Head loss through filter and well screen
h_m	Head (net) beneath top stratum midway between wells exclusive of H_w
h_o	Head (net) beneath top stratum at landside toe of levee (without seepage control measures) assuming top stratum capable of withstanding such a head
h'_o	Head beneath top stratum at landside toe of levee (measured above natural ground surface or tailwater) with a landside seepage berm
h_v	Velocity head loss in relief well
I	Percentage of imperviousness of an area expressed as a decimal
i	Upward gradient through top stratum landward of levee
i_o	Allowable upward gradient at landside toe of levee
i_1	Allowable upward gradient at toe of landside seepage berm
K	Complete elliptic integral of first kind
$\bar{K}$, $\bar{k}_f$	Coefficient of permeability of entire transformed pervious substratum
k	Coefficient of permeability
$\bar{k}_n$, $\bar{k}$	Permeability of a pervious stratum after transforming substratum to an isotropic, homogeneous stratum
k_B	Vertical permeability of artificial riverside blanket
k_{Bb}	Average combined vertical permeability of riverside natural top stratum and artificial blanket
k_{bR}	Vertical permeability of top stratum riverward of levee, particularly that in riverside borrow pits
k_f	Permeability of pervious foundation
k_H	Horizontal permeability of a pervious stratum
k_{H-n}	Horizontal permeability of individual stratum
k_v	Vertical permeability of a pervious stratum
k_{v-n}	Vertical permeability of individual stratum
L_B	Width of riverside borrow pit and/or required length of artificial riverside blanket
L_s	Length of a sublevee basin in feet measured along length of levee
L_1	Distance from riverside toe of levee to river
L_2	Base width of levee, and berm if present

L_3	Landward (effective) extent of top stratum
ΔM	Net seepage gradient toward a line of relief wells
Q	Well or seepage flow per unit length of levee
Q_B	Seepage flow into free-draining berm per unit length of levee
Q_r	Rainfall runoff into sublevee basin
Q_s	Total seepage flow (with or without wells) per unit length of levee per unit of time
Q'_s	Seepage flow into sublevee basin
Q_T	Total discharge over weir in sublevee
Q_w	Flow from a single relief well per unit of time
R	Radius of influence for a well, or maximum average rate of rainfall over an area, in inches per hour, occurring during time of concentration
r	Ratio of allowable upward gradient through top stratum at toe of levee to that at toe of seepage berm = i_o/i_1
r_w	Effective radius of a relief well
s	Distance from landside toe of levee (or berm) to effective source of seepage entry
t	Required thickness of landside seepage berm at toe of levee, and height of water in sublevee basin
W	Effective length of well screen; penetration of well screen into pervious aquifer expressed as a decimal; base width of levee in formulas for partial cutoffs
$\overline{W}$	Actual length of well screen
X	Width of landside seepage berm or sublevee basin
X_I	Width of impervious seepage berm
X_{SP}	Width of semipervious seepage berm
X_S	Width of sand seepage berm
X_P	Width of pervious seepage berm with collector system
x_r	Effective length of riverside blanket required to reduce h_o to h_a
x_3	Distance from landside toe of levee (or berm) to effective seepage exit
z_B	Thickness of artificial riverside blanket
z_{Bb}	Total effective thickness of natural and artificial riverside top stratum
z_{bL} or z_L	Effective thickness of top stratum landward of levee
z_{bR} or z_R	Effective thickness of top stratum riverward of levee, particularly that remaining in riverside borrow pit

z_t	Critical thickness of landside top stratum
θ_{av}	Average uplift factor for a line of relief wells
θ_m	Mid-point uplift factor for a line of relief wells
γ'_t	Submerged unit weight of seepage berm
γ'_z	Submerged unit weight of landside top stratum
γ_w	Unit weight of water
λ_1, λ_2	Uplift factors in formulas for design of landside drainage trenches
$\mathcal{S}$	Shape factor, the ratio in a flow net of the number of flow channels to number of equipotential drops from the seepage source to exit

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PROCEDURE FOR RAPID CONSOLIDATION TEST[a]

Closure by H. L. Su

H. L. SU,[1]—Mr. Silveira has made some very interesting comments on the paper. His table showing the correspondence between the degree of consolidation and the time factor ratios is useful. As to the incorporation of the square root fitting method, the extra time required to obtain the 90% consolidation point seems to be a negligible handicap. After all the extra time is only a matter of a few minutes.

The principal advantage of the square root fitting method is the linearity of the early portion of the time-settlement curve. The steepest slope is well defined, therefore, the secant passing through the 90% point can be precisely determined.

Mr. Fenske's observations are comprehensive. His observed variation of the slope is quite a problem. In fact, it was pointed out in the paper that a detailed discussion on the s-value had to be deferred. The reason is that such variation cannot be integrated into the slope simply as a correction at the present.

The crux of this problem lies in the fact that there is an optimum range of water contents for the validity of Terzaghi's rheological model. Within this range, his model is fairly accurate, and the s-value derived from it will be confirmed by experiments. Outside this range, the slope will vary: it will be steeper near the liquid limit and less steep around or beyond the plastic limit. This phenomenon reveals that the stiffness of Terzaghi's symbolic spring increases with decrease of the water content, and the natural conclusion will be that a clay undergoing a consolidation test does not remain as "a" clay so far as its mechanical behavior is concerned. However, most of the clays in their natural state are fortunately within the optimum range. Consequently, engineers intending to use the rapid method should not be unduly worried over this variation.

On the quantitative aspect, the s-value given in the paper, in spite of being a little on the high side, is quite fair. The discrepancy between Mr. Fenske's slope and that mentioned in the paper is possibly due to the difference in size of his specimens from that required by the British convention. (3/4 in. thick and 3 in. dia.) It is interesting to note that the slope for some natural clays, e.g. London clay and several clays from Manchester locality, is not different from that for the bentonite mentioned in the paper. This confirms the correctness of the theoretical basis for the rapid method.

As to the stiff overconsolidated clays, it must be remembered that Terzaghi's model was established under certain conditions. The nebulous

a. Proc. Paper 1729, August, 1958, by H. L. Su.

1. Engr., Ove Arup & Partners, 8 Fitzroy St., London, W. 1. Eng.

relation mentioned by Mr. Fenske must be due to the structural strength of clays, which is not included in Terzaghi's model. In fact, there is a limit of overconsolidation beyond which the soil skeleton will carry the major burden and the squeezing out of water becomes a "secondary time effect". In other words, such clays are no longer clays defined by engineers in the normal sense. It seems therefore that his tests can only be interpreted satisfactorily in terms of other rheological models.

On the question of economics, it is difficult to draw a rigorous conclusion. Every practical case must be weighted individually. Under the conditions specified by Mr. Fenske, he is right. However, when the number of tests much exceeds the number of available oedometers, the best solution seems still the rapid method even under such conditions.

In addition to the above points raised during the discussion, there is an important implication of the rapid method worth mentioning. Although the rapid test is presented primarily as a laboratory test, its application is far reaching. The immediate extension of the rapid procedure will be the site test, which is impossible by means of the conventional method.

Moreover, the purpose of the consolidation test is to provide information for the assessment of settlements and other accompanying phenomena of the prototype due to consolidation. The commonest cases are the estimation of the settlement due to the erection of engineering structures and the assessment of the geological age of clay strata. In both cases, the loading is increased gradually without any regard to whether consolidation under previous loads has surpassed the 100% point can never be reached and the clay is always partially consolidated during the process of loading. Therefore, a test procedure in which the loading sequence on the prototype can be simulated will prove to be closer to reality than the conventional test procedure.

COMPUTATION OF THE STABLIITY OF SLOPES[a]

Closure by Otto H. Meyer

OTTO H. MEYER,[1] F. ASCE.—Messrs. Sarkaria and Worth mention the versatility and rapidity of electronic computers in solving problems such as stability of slopes. Nevertheless, where a computer is not immediately available in the same office, the inconvenience in transmitting data for computation, and time loss, will usually outweigh the saving except for very extensive investigations.

In the case of the dam on a rock foundation, it is obvious that boundary conditions are a part of the problem, and must be considered, regardless of the method used. As the factor of safety is not very sensitive to the value of y, an approximate value of y may be found by the procedure in the paper; then an arc with the resulting central angle may be fitted graphically to the boundary condition. This will establish a new effective value of H, and the remainder of the solution is then straightforward.

In a zoned embankment, arcs having major portions of their lengths in the weakest materials can be drawn by inspection. Values of ϕ and c may be averaged on these arcs for reasonably approximate solutions. After all, there is little point in trying to produce solutions much more precise than the obviously approximate basic data.

Mr. Bitoun calls attention to an interesting feature of slopes of cohesionless materials. This indicates that in such cases failure will be by surface raveling rather than deep-seated slides. Where the surface of failure is the surface of the slope, the mathematical expression of that surface is of little significance.

It may also be noted that in the case of a frictionless material y will reach a maximum where: $2y = \tan y$, about $y = 66° 50'$. The factor of safety then becomes approximately $8.3c/wH$.

a. Proc. Paper 1824, October, 1958, by Otto H. Meyer.

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SETTLEMENT OF OIL STORAGE TANKS[a]

Closure by Andrew M. Braswell, Jr.

ANDREW M. BRASWELL, JR.,[1] F. ASCE.—The writer feels that the comments of the discussers add considerably to the value of his paper for which he is grateful. Many of the questions are beyond the scope of the original paper since they deal with foundation design; nevertheless, the writer will attempt to answer them to the extent of his practice and experience.

Mr. Focht inquired as to the lateral forces that caused some of the older concrete rings to fail. The writer has seen numerous instances where this has happened. In some cases, a ten-foot section of ring has been displaced laterally about two feet so that it was entirely outside the tank. Most of the rings that failed have been 24 inches square in cross-section and reinforced with two 7/8-inch round bars in the bottom, as shown in Fig. 2A. In general, the top of the rings were flush with the surface of the ground and the tanks were approximately 120 feet in diameter. Stress in the bars due to eccentric loading on the ring, with the tank full of oil, would be about 22,900 psi. For a yield point of 33,000 psi, this would leave a net stress of 10,100 psi in the bars to resist hoop tension. The combined resistance due to passive earth pressure of the soil outside the ring and the hoop tension in the bars indicates that the lateral pressure on the ring must have exceeded 900 pounds per square foot.

With regard to Mr. Focht's question concerning the calculation of lateral pressure, the writer uses the simple expression:

$p = q - 2c$, for tank rings.

where p = lateral pressure, lbs/sq. ft.

q = vertical pressure, lbs/sq. ft.

c = cohesion of soil, lbs/sq. ft.

Since the lateral pressure exerted on tank rings in this region is almost invariably that due to the top two or three feet of a very plastic clay, or gumbo, the writer feels that a more complicated analysis is not warranted. This expression, applied to the older tank rings described above, would give a lateral pressure of approximately 1,120 psf for a cohesion of 500 psf. This is the same order of magnitude as the calculated pressure necessary to cause failure of the old foundation rings.

Messrs. Brickell and Smith state that the larger a ring becomes the greater are the forces it must resist. The writer agrees that this may be

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a. Proc. Paper 1863, December, 1958, by Andrew M. Braswell, Jr.

1. Staff Eng., Humble Oil & Refining Co., Baytown, Tex.

true under certain conditions. Where the cohesion exceeds approximately one-fourth of the vertical pressure, however, the passive pressure of the soil outside the ring increases at a faster rate than the lateral pressure against the ring. For weak soils it may be necessary to extend the rings to firm soil, even though by so doing the rings must be enlarged and strengthened.

Messrs. Brickell and Smith also question the use of rigid concrete rings. Terzaghi gives the bearing value of a circular footing at the surface of a clay soil ($\phi = 0$) as:

$$q_u = 7.4\ c$$

or

$$q_a = 3.7\ c \text{ (for a safety factor of 2.)}$$

where q_u = ultimate bearing pressure, lbs/sq. ft.

q_a = allowable bearing pressure, lbs/sq. ft.

c = cohesion, lbs/sq. ft.

Therefore, a ring would not be required for a 48-foot high tank full of water if the cohesion of the clay was 810 pounds per square foot or greater. This assumes, however, that the load is uniformly distributed over the base of the tank. In reality, the shell and the portion of the roof that is supported by the shell weighs about 1,345 pounds per linear foot for a 150-foot by 48-foot tank. In the absence of some sort of foundation, this load would reach the soil essentially as a concentrated line load, which is also permanent and constant. The writer has seen numerous tanks that were built on top of a clay soil many years ago without a foundation ring. Almost without exception the shell of these tanks has settled from 6 inches to 12 inches more than the tank bottom three feet away. Core borings 1, 2, and 3 do indicate that a ring is not required at those sites, whereas Core boring 5, which is stated to be somewhat typical for this area, indicates that some sort of foundation would be required, even on the basis of a uniformly distributed load.

The writer agrees with Messrs. Brickell and Smith that, in many cases, a pad of sand or crushed stone, or a segmental concrete ring would be adequate from a soil bearing standpoint. The writer believes, however, that the additional cost of a continuous concrete ring can be justified for most installations in this area for the following reasons:

1. Sand pads and natural soil berms tend to erode from around the edge of the tank thus eliminating support for the shell.
2. Differential settlement of cone roof tanks may cause buckling of the shell or result in the roof and rafters being torn away from a portion of the shell. Appreciable differential settlement of floating roof tanks may make the roof inoperable. It has been the writer's experience that continuous reinforced concrete rings tend to bridge many local soft spots and reduce differential settlement of tanks.

d. Of perhaps minor and local importance is the function of concrete rings for tanks that are under cathodic protection. The continuous, impervious circumferential ring tends to maintain a constant moisture content in the soil beneath the tank. A certain amount of moisture is necessary for the flow of electricity in order to make this method of corrosion protection effective. This is a big objection, locally, to sand or rock cushion under tanks.

Messrs. Brickell and Smith appear to have misinterpreted Fig. 2C. It was not the intent to infer that all of the soil beneath the tank would fail simultaneously. The writer would be very surprised indeed if the soil beneath and adjacent to a large tank were so uniform that failure of the entire soil mass occurred spontaneously. The design assumes that a failure as shown in Fig. 2C might occur along a limited segment of the ring, but this failure might occur at any point on the circumference. So far as the indicated basis of design is concerned, however, it does not seem to be important whether failure is assumed to occur locally in a short section or suddenly around the entire ring. The writer has seen one instance in which failure occurred beneath a 145-foot by 48-foot tank in the manner indicated in Fig. 2C. Failure occurred during the water test when the tank contained 45 feet of water. Soft material beneath the tank for about eight feet from the edge pushed out over about a 45 degree sector tilting the concrete ring and causing a settlement of six inches. Ultimate bearing capacity of the soil was computed by Terzaghi's "Simplified-Method" to be 3,600 psi. Failure occurred at an average pressure of 3,000 psf under the ring.

The writer regrets that he cannot include settlement data for the center of any of the tanks discussed in the original paper. Only tank No. 4 has been out of service since initial installation, and the length of the downtime did not permit settlement levels to be taken in this instance. Personal obervation of many tanks in this area, however, indicates that the edge invariably settles more than the center.

Consider a 150-foot by 48-foot tank. The average depth of the contents will be 24 feet of oil with a specific gravity of 0.85. This will produce a soil pressure of 1,272 psf at the center. The weight of the shell, liquid carried by the ring, and weight of ring (in excess of the soil) will produce a soil pressure of 1,812 psf beneath the ring. When one considers that the effect of the greater soil pressure beneath the ring, as compared to the center of the tank, will be confined to the relatively more compressible surface layers, it is not too surprising that settlement at the edge exceeds that at the center of the tank.

Messrs. Brickell and Smith comment that the belly in the floor of the tank might have a greater effect upon gaging accuracy than differential settlement. The writer agrees that this might be true in those instances where a tank is emptied completely. It is extremely rare, however, that the tanks with which the writer is familiar are pumped dry. The usual procedure is to alternately pump into and out of a tank without ever completely filling or completely emptying it. Where this practice is followed, the gaging error due to differential settlement is involved each time the level in the tank is changed, whereas the error due to the configuration of the bottom is not involved until the tank is emptied.

No special precautions are taken to insure that the soil in the bottom of the excavation below the ring remains undisturbed, but it must be firm when the ring is poured. Frequently rains soften the soil after the excavation is completed, or soft spots are encountered. In these cases, the soft material must be removed and backfilled with crushed rock, sand, or compacted clay.

In conclusion, the writer wishes to make a correction to the original paper. In the column "Consolidation of Layer", under the section entitled "Comparison of Theoretical and Actual Settlements", he inadvertently neglected to divide the consolidation of some of the layers by $(1 + e_o)$. The correct values should be:

Depth	Consolidation of Layer
0-8'	2.9"
8-18	0.7
8-32	0.7
32-48	0.5
48-68	0.4
76-104	0.2
	5.4"

The calculated settlement at the edge due to consolidation of the soil is 5.4 inches. The settlement during the water test was estimated from Fig. 9 to be 2.7 inches, which gives a total calculated settlement of 8.1 inches.

PHYSICO-CHEMICAL PROPERTIES OF SOILS ION EXCHANGE PHENOMENA[a]

Discussion by J. Amorocho

J. AMOROCHO,[1] F. ASCE–Excellent summaries of the current theoretical formulations that purport to describe quantitatively the ion exchange phenomena in soils, and critical revues of these formulations are available in the literature (1, 2, 3, 4). Dr. Taylor's paper does not present one such critical revue; it would seem however, quite desirable to include a general discussion of this important subject in the Symposium on the Physico-chemical properties of soils, because it is indicative of the current state of knowledge in the field. This discussion is based on material contained in the references cited.

A very large amount of work has been done over the years with regard to cation exchange. Investigations on anion exchange have been far less abundant. In recognition of this fact, the present discussion will be confined to cation exchange only.

Many difficulties have been encountered in formulating a quantitative evaluation of ion exchange as a function of all the variable and parameters that appear to be significant in the phenomenon.

The author's paper illustrates very eloquently the large number of factors that enter the exchange and the great complexity of their interaction.

The principal approaches proposed for the quantitative evaluation of cation exchange fall within the following categories:

1. Empirical relationships based on the early gas adsorption concepts.
2. Relationships based on Mass-Action Law
3. Relationships based on the analysis of the kinetic action attendant upon surface ionization
4. Relationships based on similarity between the exchange reactions in clay systems and the Donnan osmotic equilibria
5. Relationships based on application of the theory of the diffuse double layer

All of the above have been found applicable to a limited extent only, usually within narrow experimental ranges and for rather specific cases. This is due either to the fact that all the factors involved in the exchange have not been properly accounted for in each instance, or that some of the basic underlying assumptions are unwarranted or only approximate.

Before making a short analysis of each of these general approaches, it is convenient to present in synoptic form the basic entities that operate in

a. Proc. Paper 1999, April, 1959, by A. W. Taylor.
1. Research Engr.—Univ. of California, Berkeley, Calif.

the exchange, in conjunction with the ways in which these entities find physicochemical expression in the reactions. These three basic entities are:

a) The minerals, which find expression in the reaction in terms of exchange capacity.
b) The ions, which react according to their replaceability.
c) The conditions of the reaction, which are expressed in terms of the rate of exchange.

A systematic grouping of the most important interactions that transpire from a review of published data is portrayed in the accompanying chart. (Fig. 1)

A few remarks relative to the contents of the chart are pertinent as a preamble to the discussion of the equations proposed to describe exchange reactions quantitatively:

a) The exchange capacity is determined by the structure of the mineral and may be modified by external conditions only to a limited extent.
b) The replaceability is primarily dependent on the individual characteristics of each cation but can be strongly modified by conditions of the other elements of the reaction, such as the concentration of the replacing solution and the structure of the clay.
c) Finally, the influence of the general conditions of the reaction in manifested directly in the rate of exchange. This rate is also influenced simultaneously by properties of the clay and of the ions.

Let us examine the various relationships currently available:

1. Empirical relationships.

Under this group, a number of equations have been proposed (Langmuir, Wiegner, Jenny, Vageler) which involve various modifications of Freundlich's gas adsorption concept as applied to solution ions. In principle, these equations express a simplified state of equilibrium between the total amount of cations adsorbed by a solid (in this case the soil) and the amount of cations remaining in solution. In order to satisfy the mathematical equalities, a number of empirical constants are introduced, which presumably account indirectly for the individual properties of the soils and/or the ions.

Typical of this group is the Wiegner-Jenny equation

$$a - c = k\left(\frac{c}{a - c}\right)^{1/p}$$

where $a - c$ = amount of cations adsorbed per gram of soil

c = concentration of original cations remaining in solution at equilibrium

a = initial concentration of the solution

k and p = constants

The following comments may be presented on this expression:

a) The formula presupposes a continuous functional relationship between the amounts of ions adsorbed and the original concentration of the solution. The values of k and p, which are related to the ionic radii and, presumably to the character of the soil, are kept constant. The weakness of these premises becomes evident by considering the relationships shown in the synoptic Chart (Fig. 1). It is seen that factors dependent on the ambient conditions of the reaction, as well as changeable influences determined by the

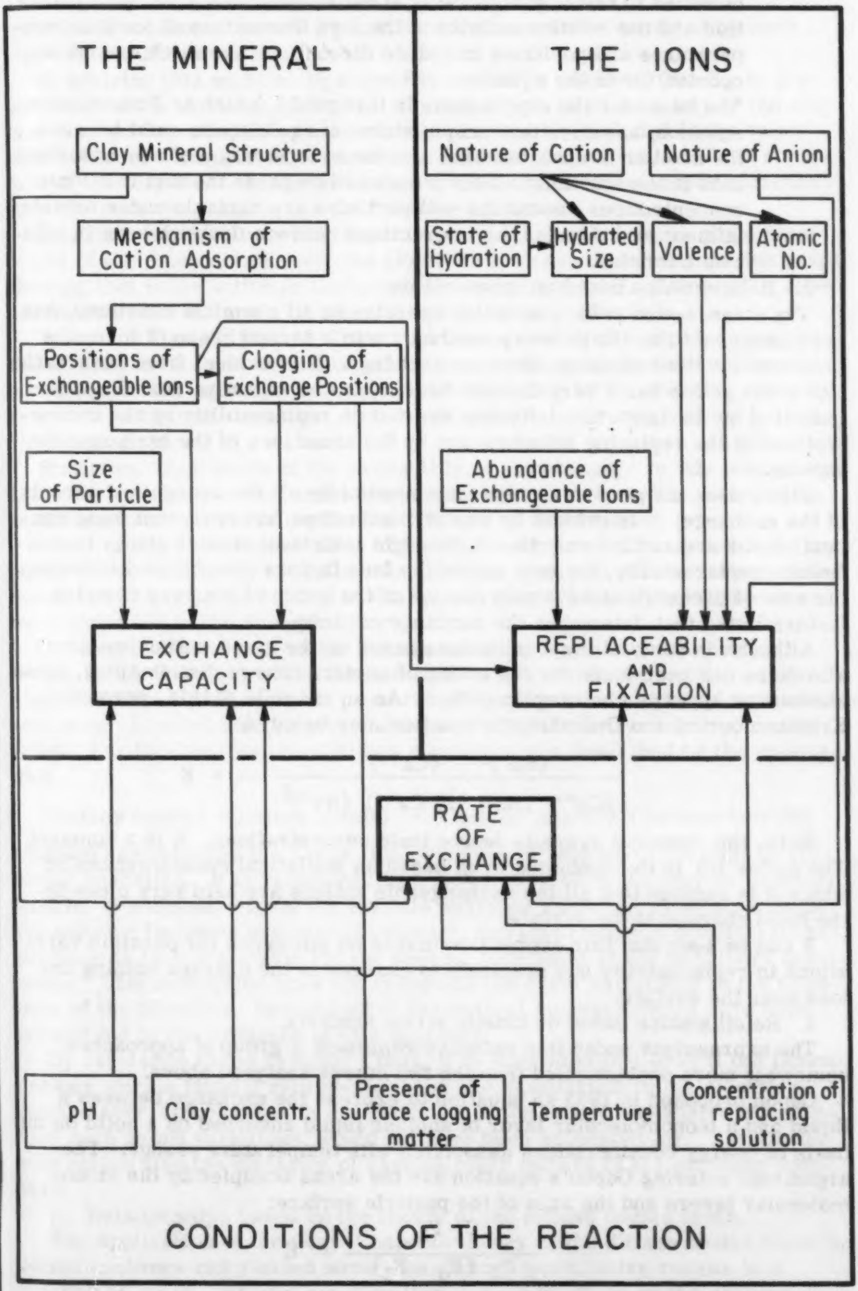

FIG. I SYNOPTIC CHART OF THE CATION EXCHANGE

behavior of the clay structures at different states of surface saturation and the relative activity of the ions themselves at various temperatures and positions introduce discontinuities which are not accounted for in the equation.

b) The basis for the expressions in this group, which is Freundlich's equation, assumes that unique states of equilibrium exist between the number of adsorbed ions and the concentration of non-adsorbed ions in the solutions. This premise disregards the fact that ionic concentrations around the soil particles are variable under outside influences and under the interactions between double layers in adjacent particles.

2. Relationships based on mass-action.

The mass-action principle, which operates in all chemical reactions, has been assumed to be the primary mechanism in a second group of formulas proposed for the evaluation of cation exchange. It is evident from observation that mass action has a very definite bearing on the exchange reactions, as indicated by the important influence exerted on replaceability by the concentrations of the replacing solutions and by the abundance of the exchangeable cations.

Fig. 1 does not purport to represent absolutely all the recognized aspects of the exchange; it is evident by way of illustration, however, that ionic concentrations account for only two of the eight individual factors shown that influence replaceability, for only one of the four factors listed that determine the rate of exchange and for only one out of the group of six very complex factors listed that determine the exchange capacity.

Although in some of the expressions based on the mass-action law some allowance has been made for the action of surface charge distributions, these allowances appear to be oversimplified. As an example of this approach, Krishnamoorthy and Overstreet's equation may be cited:

$$\frac{(Na^+)^2 \qquad (Ca^{++})}{(Ca^{++})\ (Na^+\ \ 1.5\ Ca^{++})\ (Na^+)^2} = K$$

Here, the chemical symbols denote ionic concentrations. K is a constant. The factor 1.5 in the denominator is based on statistical considerations in which it is implied that all the exchangeable cations are held very close to the fixed charges at the surface.

It can be seen that this expression makes no allowance for possible variations in replaceability due precisely to changes in the charges holding the ions near the surface.

3. Relationships based on kinetic action analysis.

The expressions under this category represent a group of approaches somewhat more sophisticated than the two others analyzed above.

Gapon proposed in 1933 an equation to express the exchange between a liquid and a monomolecular layer of another liquid adsorbed on a solid on the basis of energy considerations associated with temperature change. The arguments entering Gapon's equation are the areas occupied by the monomolecular layers and the area of the particle surface:

$$\frac{C_1\ F_2}{C_2\ (F_o - F_1)} = K$$

Where C_1 and C_2 denote the two liquids, F_0 is the total area of particle involved in the exchange and F_1 and F_2 are the surfaces occupied by each liquid at equilibrium.

In applying this equation to a specific case it is assumed that F_0 is proportional to the exchange capacity. Therefore this basic property of the clay is accounted for. Cationic constants are presumably included in the value of K. However, complete abstraction is made of the properties of the structure of the clay that actually produce discontinuity of the adsorption intensity, such as those responsible for the fixation of cations.

It is interesting to note that Gapon's basic equation takes the same form of one of the mass action formulas (Kerr) in the case of monovalent ion pairs, showing that mass action is implicit in Gapon's treatment. However, it does not consider the influence of the clay concentration. (See Fig. 1).

Jenny and Davis (1936, 1945) utilized the concept that each exchangeable cation oscillates about the center of an electrical charge in the surface of a particle and that exchange takes place when a cation of the solution, which is in a state of thermal agitation, chances to substitute the original ion in the electrical field.

Statistical treatments of the probability of the exchange in this mechanism give rise to equations that define the exchange constant used in mass action equations in terms of the oscillation volumes of the cations.

Here, again, the concentration of the clay is ignored. It is noted that Davis' equation does not apply except within a narrow range of concentrations of the solution. This may be related to changes in the replaceability of the cations as is indicated in the Synoptic Chart.

4. Expressions based on the Donnan osmotic equilibrium concept.

The original theory of the Donnan equilibrium states the conditions of electric balance necessary to preserve neutrality at both sides of a membrane enclosing colloidal electrolytes and separating them from ordinary electrolytes. For this system, equilibrium conditions are described by the expression

$$\text{(Anions inside) (Cations inside)} = \text{(Anions outside) (Cations outside)}$$

Mattson has extended the use of this principle by analogy to the case of colloidal miscelles. It is reasoned that a fictitious membrane may be imagined to exist at a distance from the particle surfaces where the concentration of the solution becomes essentially constant, and that Donnan's equilibrium expression is then applicable for the ions on both sides of the imaginary membrane. This conception does not recognize the great variability of the thickness of the micelles. Several other theoretical inconsistencies have been pointed out in the critiques of this approach.

By reference to the Synoptic Chart, it is suggested that even if the Donnan analogy were a faithful representation of the equilibrium in the electrical double layer, the changes in the thickness of the latter, associated with factors such as pH variations, would not have been accounted for. The reader may notice also the omission of various other factors of the exchange complex.

5. Relationships based on the theory of the diffuse double layer.

The application of the Gouy-Chapman theory of the diffuse double layer to cation exchange has yielded some satisfactory quantitative results in a number of cases. Without question this method of attack on the problem incorporates a more advanced knowledge of the ionic actions than the ones

previously discussed. However, as pointed out by Low[4], some inaccuracies in the assumptions underlying the double layer theory tend to vitiate the accuracy of the predictions based upon it.

CONCLUSION

The present state of knowledge of the different aspects of the cation exchange phenomena, as they have been observed in the very large number of experimental studies conducted since the exchange was first recognized, has made it quite evident that there exists a gap between the large volume of facts of which the workers in the field are aware and the postulation of expressions adequate to permit their evaluation. The application of the Gouy-Chapman Theory of the diffuse double layer represents a major advance in the direction of a unified theory of the cation exchange; however, a systematic grouping of the variables and parameters, such as is shown in Fig. 1 of this discussion, suggests that representation of a phenomenon of the formidable complexity of ion exchange by a single formula is not logical. In support of this contention it may be pointed out that several of the partial cause-and-effect relationships that appear to exist in the exchange are not continuous functions of the same parameters. This being the case, the only possible way to evaluate ion exchange would be to treat it as a system of related functions rather than as a single function. This approach would permit the independent solution of the intervening cause-and-effect components before synthesizing the final result. This is contrasted to the idea of interlocking in a single expression all the independent and interdependent variables. The difficulty of the latter course of action may be illustrated by the following simplified analysis of some of the experimental findings that are not accounted for in the double-layer theory, as an example.

It is known, for instance, that the hydrogen ion concentration, as pointed out in the paper, produces important variations in the exchange capacity of the clays. Mathematical expressions could conceivably be devised to represent the pH vs. exchange capacity function over a wide range of values for any particular clay mineral. It is also known, however, that the inflections shown by the titration curves at extreme pH values are due to partial disintegration of the clays associated with the attack of the mineral by the acids of the alkalies of the medium. Obviously at these stages profound changes take place in the basic elements which, being of a nature entirely foreign to the exchange, could hardly be introduced in a generalized expression for the latter. Other discontinuous effects that have been discovered by experimental observation are those related for instance to size. Although size is not a very important parameter in minerals like montmorillonite, in which cation adsorption due to broken bonds is secondary, continued particle subdivision such as that produced by grinding is finally felt in these layered clays and expressed as a rather sudden increase in exchange capacity.

In conclusion, it is believed that caution should be exerted at the present state of knowledge of cation exchange in order to avoid the danger of mathematical oversimplifications. It is suggested that a systematization of the quantitiative analysis of secondary phenomena in the exchange might be fruitful in solving the overall problem.

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PHYSICO-CHEMICAL PROPERTIES OF SOILS: ROLE OF SOIL TECHNOLOGY[a]

Discussion by A. A. Eremin

A. A. EREMIN,[1] M. ASCE—The most interesting minerals which may occur in clayey soils are montmorillonite and bentonite. In his diagram Mr. Lambe has shown the influence of montmorillonite content on the plastic properties of clayey soils. It would be interesting to express expanding properties of clayey soils containing montmorillonite or bentonite in the quantitative form.

It is interesting to note that after drying and again wetting the soil containing the montmorillonite expanding properties of soil are generally resumed. Furthermore, after stirring of moist soil containing montmorillonite expanding properties of soil are accelerated.

In the analysis of soil for montmorilonite content it should be remembered that montmorillonite occurs only in geological formations not older than the mesozoic period.

Analysis of physical and plastic properties of soils is important not only in investigating bearing properties of foundation material but also for commercial use of clayey soils. Therefore, Mr. Lambe should be highly acknowledged for his interesting paper giving valuable informations on the technology of clayey soils.

a. Proc. Paper 2001, April, 1959, by A. A. Eremin.

1. Associate Bridge Engr., California State Highway, Bridge Dept., Sacramento, Calif.

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PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

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OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806(HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815(ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824(SM4), 1825(SM4), 1826(SM4), 1827(ST6)c, 1828(SM4)c, 1829(HW3)c, 1830(PO5)c, 1831(EM4)c, 1832(HY5)c.

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850 (SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)c, 1856(HY6)c, 1857(ST7)c, 1858(SU3)c.

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)c, 1881(SM5)c, 1882(ST8)c, 1883(PP1)c, 1884(WW5)c, 1885(CP2)c, 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

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JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)c, 1917(EM1)c, 1918(HW1)c, 1919(HY1)c, 1920(PL1)c, 1921(SA1)c, 1922(ST1)c, 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)c, 1951(SM1)c, 1952(ST2)c, 1953(PO1)c, 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)c, 1986 (IR1)c, 1987(WW1)c, 1988(ST3)c, 1989(HY3)c.

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HW2)c, 2008(EM2)c, 2009(ST4)c, 2010(SM2)c, 2011(SM2)c, 2012(HY4)c, 2013(PO2)c.

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)c, 2041(PL2)c, 2042(PP1)c, 2043(ST5)c, 2044(SA3)c, 2045(HY5)c, 2046(PP1), 2047(PP1).

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JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087 (SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096 (EM3), 2097(HY7)c, 2098(SA4)c, 2099(EM3)c, 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134 (SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)c, 2139(PO4)c 2140(SM4)c.

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)c, 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158 (IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176 (WW3), 2177(WW3), 2178(CO2)c, 2179(IR3)c, 2180(HW3)c, 2181(SA5)c, 2182(HY9)c, 2183(SU1)c, 2184 (WW3)c, 2185(PP2)c, 2186(ST7)c, 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205 (PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214 (SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223 (ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)c, 2233(PL3)c, 2234(EM4)c, 2235(HY10)c, 2236(SM5)c, 2237(ST8)c, 2238(PO5)c, 2239(ST8), 2240 (PL3).

c. Discussion of several papers, grouped by divisions.

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NO. SM 5
PART 2

Your attention is invited

NEWS OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION OF ASCE

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

October, 1959

SOVIET CONTRIBUTIONS TO SOIL ENGINEERING SEMINARS IN THE U. S.

Professor Gregory P. Tschebotarioff, Professor of Civil Engineering at Princeton University has forwarded the following news items regarding the recent visit of the Soviet soil mechanics and foundations engineers to the United States.

The Soviet delegation left the United States on June 22 after a threeweeks stay. Their arrival and the Soviet vibratory pile driving methods described by them at the first (Princeton) Seminar were reported earlier (Civil Engineering, July 1959, p. 69). A number of interesting contributions on other topics were also made by them during their visit.

Thus, Professor N. A. Tsytovich outlines at the Princeton and M.I.T. Seminars Societ theory and practice of foundation construction on frozen ground, a field in which much pioneering work has been done in the U.S.S.R. Professor F. J. Sanger, Mr. R. R. Philippe and other engineers of the U. S. Corps of Engineers participated in the friendly discussion which followed.

Professor Tsytovich gave two other reports—one (at M.I.T. and in Washington, D. C.) on the historical background and present development trends of soil mechanics work in the Soviet Union; the other (at Illinois) on Soviet methods of foundation design based on limit states of soil equilibrium and on limit settlement values permitted for different types of structures.

The chairman of the delegation, Academician I. M. Litvinov, gave two reports. The first (at M.I.T. and at Illinois) dealt with Soviet work on thermal stabilization of loess soils in the Ukraine. The structure of such macroporous soils easily collapses when they are rain-soaked under load. By injecting compressed air, together with fluid or gas fuel directly into bore holes through special burners and perforated casings, the loess was transformed to a depth of 30 feet around each hole into a 10-foot diameter cylinder of hard brick-like substance. The method is economical for large structures and shallow foundations. It was successfully used under new multi-story buildings and 300' high factory chimneys. It was even used for underpinning since no

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shrinkage of the baked loess occurs insitu, presumably due to pressures which the combustion gases and evaporating water develop in the pores of the loess.

The second report by I. M. Litvinov (at California) described Soviet field sampling and field laboratory soil testing procedures.

Mr. I. M. Levkin repeated at Illinois and at California his Princeton report on the Soviet use of heavy vibrators for the sinking of piles and large (up to 16' diameter) open-end cylindrical pre-cast reinforced concrete caissons.

Mr. R. A. Tokar outlined (at Princeton and at M.I.T.) the Soviet approach to machine foundation design.

Professor V. M. Bezruk reported (at California and in Washington, D. C.) on the Soviet procedures for flexible pavement design.

The Soviet engineers presented their American local hosts with copies of many Russian language publications which amplified the topics dealt with in their Seminar reports.

Other chairmen of the local committees for the Seminars were: Professor T. W. Lambe at M.I.T.; Professor R. B. Peck (Illinois) and Professor G. A. Leonards (Purdue) at Illinois; Professor H. B. Seed at California. Mr. Fred Burggraf, Director of the Highway Research Board, organized the Seminar at the National Academy of Sciences in Washington, D. C. and the committee for the entire exchange, of which committee Professor M. S. Kersten of Minnesota was chairman.

The Highway Research Board is preparing the publication of translations of the Soviet reports with the relevent illustrations shown at the five U. S. Seminars. The Board was the prime sponsor of this exchange of delegations, which was co-sponsored by the ASCE.

In addition to the laboratories of the four universities where the Seminars were held, the Soviet delegation visited the soil engineering laboratories of Rutgers and Harvard Universities, of the California Division of Highways at Sacramento and of the Bureau of Public Roads in Washington D. C. The ASSHO test road at Ottawa, Illinois and foundation construction sites in New York, Boston and Chicago were also visited.

COLORADO RESEARCH CONFERENCE on SHEAR STRENGTH OF COHESIVE SOILS

Plans for the Research Conference on Shear Strength of Cohesive Soils, to be held in Boulder, Colorado, in June 1960 have now taken more concrete form. The conference is sponsored by the Soil Mechanics and Foundations Division, ASCE and a definite program will later be mailed to members of this division and to all others who may request it. In the process of completing all arrangements, minor changes may be made in the plans as summarized below.

The Colorado Section, ASCE, and the University of Colorado will be hosts to the conference. Dr. W. J. Turnbull is Chairman of the Task Committee on Shear Strength of Soils, which is charged with organization of the conference, and Mr. W. G. Holtz is Chairman of the Local Committee on Arrangements.

Inquiries concerning attendance at the conference should be directed to Dr. J. W. Hilf, Secretary of the Task Committee on Shear Strength of Soils, Bureau of Reclamation, Building 53, Denver Federal Center, Denver 25, Colorado.

Purpose of the Conference

The general purpose of the conference is to assemble, summarize, and discuss present knowledge, or lack of knowledge, concerning the factors which govern the shear strength and failure conditions of cohesive soils.

Place and Time of Conference

The conference will be held at the University of Colorado in Boulder, Colorado, on June 13-17, 1960. Social activities as well as technical sessions are planned.

Participation in Conference

Anyone in the United States and other countries may attend the conference.

Presentation of papers is by invitation only. A number of universities and govermental and private engineering organizations in the United States and other countries, whose members have performed significant research on shear strength of soils, have been invited and have agreed to prepare papers for the conference.

Authors, or their organizations, will furnish preprints of their papers. These preprints will be forwarded, as long as the supply lasts, to those who register for the conference.

Schedule of Sessions and Other Activities

The following technical sessions are planned:

Session 1. Opening addresses and failure hypotheses.
Session 2. Testing equipment techniques and errors.
Session 3. Shear strength of saturated remolded clays.
Session 4. Shear strength of undisturbed cohesive soils.
Session 5. Shear strength of compacted cohesive soils.
Session 6. Problems associated with practical applications.
Session 7. Moderators' reports, discussions, and closing addresses.

A supplementary program including tours for entertainment of conference attendees and ladies accompanying them is in course of preparation.

Conduct of Sessions

With the exception of a paper on failure hypotheses, scheduled for presentation at Session 1, preprinted papers will not be presented orally at the conference.

Sessions 2 through 6 will primarily be conducted as panel discussions. A moderator, assisted by an associate moderator, will be in charge of each of these sessions. The moderator will present a summary of preprinted papers dealing with the topic of the session and then guide discussions among panel members.

The moderator may also ask for opinions of members of the audience and may consider questions submitted in writing before the session. If time permits, the last part of each session may be opened to discussion from the floor.

Each moderator, assisted by the associate moderator, will prepare a brief summary of his session for presentation at Session 7. The moderators will also prepare more detailed reports for publication in the proceedings of the conference.

Anyone may prepare written discussions for publication in the proceedings. Such discussions should comply with the ASCE requirements and should be submitted not later than September 1, 1960.

Conference Proceedings

All papers, written discussions, authors' closures, and moderators' reports will be published in the proceedings of the conference. A copy of the proceedings will be furnished all those registering for the conference and will be sold to others.

Registration Fees

A registration fee of $15 will entitle the registrant to attend the conference and to receive a set of preprints of the papers and a bound copy of the conference proceedings. A small conference fee will be charged by the University for the use of extracurricular facilities. There will be no fees for persons accompanying registrants.

Accommodations in Boulder

A residence hall at the University will be made available at nominal costs. Excellent hotel and motel facilities will also be available. It is expected that many of the participants will combine attendance at the conference with a vacation for the entire family in Colorado where ideal climate and vacation facilities exist.

Registration of Intent

A brochure announcing the conference and giving details of the technical and social events is being prepared and will be distributed to all members of Soil Mechanics and Foundations Divisions in the near future. The brochure will contain forms for advance registration and indication of interest in attending the conference or securing a copy of its proceedings.

CIVIL ENGINEERING AND GEOLOGY AT THE UNIVERSITY OF NORWAY

Mr. George A. Kiersch, member of the Geological Society of America sends the following item of interest to men in our division. We are happy to pass it on to you and hope that we will receive more like it.

"During May and June, 1959, I spent some three weeks in Norway on geological work connected with a series of proposed construction sites. While there on these engineering geology investigations, I presented a series of eight lectures to the civil engineering students and some faculty of the Technical University of Norway, Trondheim on Engineering Geology and its practice in the United States. The lectures covered a broad introduction, followed by a review of specific geologic phenomena or problems and the manner in which treated in engineering practice in the United States for such as: engineering seismology, landslides, weathering of rocks, particularly limestone, influence of structural features on field and laboratory results and the engineer-geologist team in practice. A summary of many of the problems under investigation in the construction materials field was presented, and likewise, a few of the "new" fields now developing, such as applications of nuclear blasting, geothermal sources of energy and the growing importance of subsidence as a problem in practice.

From a review of the training offered the civil engineering students at the Technical University, they are given a broader background in geology than most schools offer in the United States. For instance, a civil engineering student majoring in a branch of the profession dealing with earthworks and foundations is required to take a two year course in geology, a beginning year of general physical geology, with some applications and a second year (senior) of advanced geology dealing with engineering applications and additional theory. The Norwegian students and faculty in civil engineering expressed a keen intereat in engineering geology, and this feeling is likewise held by the practicing engineer."

HONORARY DEGREE AWARDED TO PROFESSOR G. P. TSCHEBOTARIOFF

Professor Gregory P. Tschebotarioff of Princeton has been awarded the degree "Docteur honoris causa" of The Université Libre de Bruxelles. The insignia will be presented to him November 20, during the 125th anniversary celebrations of the University at Bruxelles.

In September, 1958 Dr. Tschebotarioff attended by personal invitation, and served as vice president, the European regional conference on Lateral Earth Pressures organized by the Belgian Group of the International Society of Soil Mechanics and Foundation Engineering.

PENNSYLVANIA SOILS CONFERENCE

The Central Pennsylvania Section of the Society is planning a Soils Conference to be held in Harrisburg the evenings of October 1st, 6th and 8th.

Speakers will include representatives of the Highway Research Board, the Bureau of Public Roads, the American Association of State Highway Officials and the Portland Cement Association.

Those interested in attending should contact Mr. H. B. Henry, P.O. Box 618, Harrisburg, Pennsylvania for further details.

Early Transactions Volumes Obtainable

The feasibility of reproducing the first ten volumes of ASCE Transactions (1872-1881) has been studied. It has been decided that these historic volumes could be reproduced at a cost that would permit a top price of $150 for the ten-volume set. If more than 100 engineers, or libraries, indicate an interest in obtaining such a set, the project will be undertaken. If the endeavor is successful, other rare volumes of Transactions will be reprinted.

Engineers interested in obtaining the ten-volume set should write to the Executive Secretary of ASCE, 33 West 39th Street, New York 18, N. Y.

DECEMBER NEWSLETTER

Deadline date for arrival at this office of contributions for the December Newsletter: October 20, please.

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